ADDIS ABABA UNIVERSITY

ADDIS ABABA INSTITUTE OF TECHNOLOGY

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

RETROFITING OF REINFORCED CONCRETE BEAMS USING FIBRE REINFORCED POLYMER

A Thesis Submitted to the School of Graduate Studies of Addis Ababa University in Partial Fulfillment of the Requirements for the Degree of Masters of Science in Structural Engineering.

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Advisor: Dr. Ing- Adil Zekaria

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Declaration

I, the undersigned, declare that this thesis work, to the best of my knowledge and belief, is my original work, has not been presented for a degree in this or any other universities, and all sources of materials used for the thesis work have been fully acknowledged.

Ramin Meried ____________________

Candidate **Signature**

Date of Submission: April 15, 2016

This is to certify that the above declaration made by the candidate is correct to the best of my knowledge.

Dr.Ing-Adil Zekaria ____________________

Thesis Advisor **Signature**

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ABSTRACT

Infrastructure development is raising its pace. Many reinforced concrete and masonry buildings are constructed annually around the globe. With this, there are large numbers of them which deteriorate or become unsafe to use because of changes in use, changes in loading, change in design configuration, inferior building material used or natural calamities. Concrete is a material that can withstand compressive loads very well but is sensitive to tensile forces. Therefore, concrete structures are typically reinforced by casting in steel bars in areas where tension can arise. This cannot be done afterwards, and one strengthening method, is therefore to glue reinforcement on the exterior of the structure in the areas exposed to tension. Fiber composite can be used in reinforcing concrete structures externally. Fiber composite materials have low density, can be easily installed and are easy to cut to length on site. Therefore, fiber composite as external reinforcement for concrete structures has become very attractive and popular around the world. Thus repairing and retrofitting structures for safe usage of these structures has a great market. There are several situations in which a civil structure would require strengthening or rehabilitation due to lack of strength, stiffness, ductility and durability. Beams, columns, plates may be strengthened in flexure through the use of FRP bonded to their tension zone using epoxy as a common adhesive. Due to several advantages of fiber wrapping over conventional techniques used for structural repair and strengthening, the use of FRP has becoming popular.

This paper makes a comparative study between the load carrying capacity of an RC beam and other beams with FRP bonded. An experiment study is carried out to study the change in the structural behavior of RC beams externally bonded with FRP of different thickness to enhance the flexural capacity of beams along with the existing practice of doing the repair work.

Experimental tests were performed to investigate the behavior of retrofitted beams. The model was verified through comparison with the experimental data regarding failure mode and load-displacement behavior.

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Notations

A_f Cross sectional area of FRP.

A_s Cross sectional area of steel.

E_f Elastic modulus of FRP.

Es Elastic modulus of steel.

F_c Compression force in concrete.

f'c The characteristic compressive cylinder strength of concrete at 28 days.

F_f Tensile force in FRP.

Fs Tensile force in steel.

Ga Shear modulus of the adhesive layer.

M Bending moment at the cutoff point.

Mcr Bending moment causing cracking.

M_d Moment capacity for the member.

H Depth from the extreme compression fibre to the externally bonded FRP

t_f FRP thickness.

x Depth of the neutral axis from the extreme compression fibre.

ε_0 Concrete strain on tension side at time of the FRP application.

ε_c Concrete strain in the extreme compression fibre.

ε_{cu} Compression failure strain of concrete.

ε_f FRP strain

1. Introduction

1.1 Background

Reinforced concrete structures often have to face modification and improvement of their performance during their service life. The main contributing factors are change in their use, new design standards, deterioration due to corrosion in the steel caused by exposure to an aggressive environment and accident events such as earthquakes.

In such circumstances there are two possible solutions: replacement or retrofitting. Full structure replacement might have determinate disadvantages such as high costs for material and labor, a stronger environmental impact and inconvenience due to interruption of the function of the structure e.g. traffic problems. When possible, it is often better to repair or upgrade the structure by retrofitting.

In the last decade, the development of strong epoxy glue has led to a technique which has great potential in the field of upgrading structures. Basically the technique involves gluing steel plates or fiber reinforced polymer (FRP) plates to the surface of the concrete. The plates then act compositely with the concrete and help to carry the loads.

FRP can be convenient compared to steel for a number of reasons. These materials have higher ultimate strength and lower density than steel. The installation is easier and temporary support until the adhesive gains its strength is not required due to the low weight. They can be formed on site into complicated shapes and can also be easily cut to length on site.

There is a considerable number of existing reinforced concrete structures that do not meet current design standards because of inadequate design and/or construction or need structural upgrading to meet new seismic design requirements. Retrofitting of flexural concrete elements is traditionally accomplished by externally bonding steel plates to concrete. Although this technique has proved to be effective in increasing strength and stiffness of reinforced concrete elements, it has the disadvantages of being susceptible to corrosion and difficult to install.

Recent development in the field of composite materials, together with their inherent properties, which include high specific tensile strength, good fatigue resistance, corrosion resistance and ease of use make them an attractive alternative to steel plates in the field of repair and strengthening of concrete elements.

The effectiveness of using Fiber Reinforced Composites in increasing strength and stiffness of reinforced concrete flexural elements is evident from results of previous research work (Bazaaet al.1996, Chajes et al. 1995, Grace. Et al. 1999 (a&b), Swamy et al. 1995) and observed behavior of field applications (ACI 440-1996, Meier et al. 1995).

Although this material has superior properties which include very high tensile strength accompanied with a reasonable modulus of elasticity (almost equals that of steel); its high cost is still one of the main disadvantages, which limits its use as a replica to externally bonded steel plates in retrofitting applications. On the other hand, the Glass Fiber Composites (GFC) are comparatively cheap and have high tensile strength but with relatively low modulus of elasticity (about one-third that of carbon and reinforcing steel). Although, GFC seem to be attractive and cost-efficient for retrofitting flexural reinforced concrete elements, few research work were conducted to study the behavior of such systems. When using GFC to retrofit flexural reinforced concrete elements, careful attention should be paid not only to the strength of the retrofitted element but also to its stiffness and ductility. Moreover, the stage of loading at which the retrofitting is carried out is one of the important aspects that impact the effectiveness of the retrofitting process.

Figure 1 Bridge in the Al Zarqa-Jordan.

In the last decade, the development of strong epoxy glue has led to a technique which has great potential in the field of upgrading structures. Basically the technique involves gluing steel plates or fiber reinforced polymer (FRP) plates to the surface of the concrete. The plates then act compositely with the concrete and help to carry the loads.

Each material has its specific advantages and disadvantages. Steel plates have been used for many years and are very effective to use as bonding reinforcement. However, they are heavy to transport and install, prone to corrosion and delivery length of plates are limited.

FRP can therefore be convenient compared to steel. These materials have higher ultimate strength and lower density than steel. The installation is easier and temporary support until the adhesive gains its strength is not required due to the low weight. They can be formed on site into complicated shapes and can also be easily cut to length on site.

Debonding is a major problem in structures retrofitted with FRP. Debonding implies complete loss of composite action between concrete and FRP. This prevents full utilization of the FRP-concrete system and may lead to failure before the design load is reached. Debonding due to a stress concentration may initiate either at the plate end or around cracks.

This work is a study of the behavior of concrete beams retrofitted with FRP, using experiments and finite element modelling.

1.2 Objectives

General Objective

The overall all aim of this research is to investigate the behavior of reinforced concrete beams retrofitted with FRP.

Specific Objective

- To evaluate and analyze the flexural capacity of RC beams strengthened with externally bonded FRP reinforcement.
- To introduce new material to the market, the material which is fabricated for plaster reinforcement but it will be checked as if it can be used for structural use.

1.3 Statement of the problem

Many reinforced concrete and masonry buildings are constructed annually around the globe; however structures show failures due to load increment and environmental impacts. Thus there is a need of strengthening such structures rather than replacing them in a new structure.

1.4 Methodology

Over the study of retrofitting concept mainly experimental work is taken and numerical analysis is done using ANSYS. Total of twelve beams are tasted, among these three beams as a control beam and nine beams as a retrofitted beam. Retrofitted beams are grouped based on number of layers of bonded FRP. Each group having three beams are tested with one, two and three layer of FRP. Afterwards analytical analysis is done with the help of ANSYS for the control and retrofitted beam. Finally load-deflection response of experimental Vs analytical result is plotted and comparison is made.

1.5 Organization of the thesis

The thesis is organized into seven chapters. Chapter1 introduces the background, the objectives, and Methodology of the thesis work. Chapter 2 describes previous works done on the retrofitting of RC beams. Chapter 3 & 4 shows the experimental program of the thesis work its result and discussion. Chapter 5 concerns the numerical investigation of this work. Finally chapter 6 &7 deals the comparison of analytical and experimental work; conclusion and recommendation are shown briefly.

2. Literature Review

2.1 General

Fiber-reinforced polymer (FRP), also Fiber-reinforced plastic, is a composite material made of a polymer matrix reinforced with fibers. The fibers are usually glass, carbon, or aramid, although other fibers such as paper or wood or asbestos have been sometimes used. The polymer is usually an epoxy, vinyl ester or polyester thermosetting plastic, and phenol formaldehyde resins are still in use. FRPs are commonly used in the aerospace, automotive, marine, and construction industries.

Fiber reinforced polymer composites consist of high strength fibers embedded in a matrix of polymer resin as shown in Figure 2.

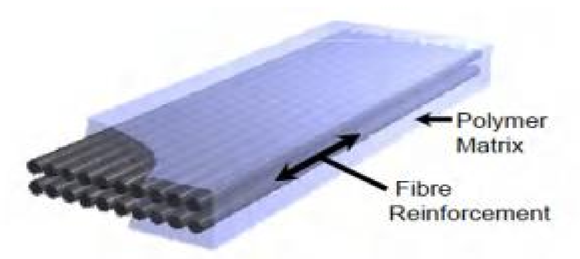


Figure 2 Combination of fibres and matrix to form an FRP composite [1].

In FRP, the fibres provide both load carrying capacity and stiffness to the composite while the matrix is to ensure sharing of the load among fibers and to protect the fibers themselves from the environment. Most FRP materials are made of fibres with high strength and stiffness, while their strain at failure is lower than that of the matrix.

Composite materials are engineered or naturally occurring materials made from two or more constituent materials with significantly different physical or chemical properties which remain separate and distinct within the finished structure. Most composites have strong, stiff fibers in a matrix which is weaker and less stiff. Commercial material commonly has glass or carbon fibers in matrices based on thermosetting polymers, such as epoxy or polyester resins.

Sometimes, thermoplastic polymers may be preferred, since they are moldable after initial production. There are further classes of composite in which the matrix is a metal or a ceramic. For the most part, these are still in a developmental stage, with problems of high manufacturing costs yet to be overcome [2].

Furthermore, in these composites the reasons for adding the fibers (or, in some cases, particles) are often rather complex; for example, improvements may be sought in creep, wear, fracture toughness, thermal stability, etc [3].

Fiber reinforced polymer (FRP) are composites used in almost every type of advanced engineering structure, with their usage ranging from aircraft, helicopters and spacecraft through to boats, ships and offshore platforms and to automobiles, sports goods, chemical processing equipment and civil infrastructure such as bridges and buildings.

The usage of FRP composites continues to grow at an impressive rate as these materials are used more in their existing markets and become established in relatively new markets such as biomedical devices and civil structures. A key factor driving the increased applications of composites over the recent years is the development of new advanced forms of FRP materials. This includes developments in high performance resin systems and new styles of reinforcement, such as carbon nano tubes and nano particles [4].

The fiber reinforced polymer composites (FRPs) are increasingly being considered as an enhancement to and/or substitute for infrastructure components or systems that are constructed of traditional civil engineering materials, namely concrete and steel. FRP composites are lightweight, no-corrosive, exhibit high specific strength and specific stiffness, are easily constructed, and can be tailored to satisfy performance requirements. Due to these advantageous characteristics, FRP composites have been included in new construction and rehabilitation of structures through its use as reinforcement in concrete, bridge decks, modular structures, formwork, and external reinforcement for strengthening and seismic upgrade [5].

The applicability of Fiber Reinforced Polymer (FRP) reinforcements to concrete structures as a substitute for steel bars or prestressing tendons has been actively studied in numerous research laboratories and professional organizations around the world. FRP reinforcement offers a number of advantages such as corrosion resistance, non-magnetic properties, high tensile strength, lightweight and ease of handling. However, they generally have a linear elastic response in tension up to failure (described as a brittle failure) and a relatively poor transverse or shear resistance. They also have poor resistance to fire and when exposed to high temperatures. They lose significant strength upon bending, and they are sensitive to stress. Moreover, their cost, whether considered per unit weight or on the basis of force carrying capacity, is high in comparison to conventional steel reinforcing bars or prestressing tendons. From a structural engineering viewpoint, the most serious problems with FRP reinforcements are the lack of plastic behavior and the very low shear strength in the transverse direction.

Such characteristics may lead to premature tendon rupture, particularly when combined effects are present, such as at shear-cracking planes in reinforced concrete beams where dowel action exists. The dowel action reduces residual tensile and shear resistance in the tendon. Solutions and limitations of use have been offered and continuous improvements are expected in the future. The unit cost of FRP reinforcements is expected to decrease significantly with increased market share and demand. However, even today, there are applications where FRP reinforcements are cost effective and justifiable.

Such cases include the use of bonded FRP sheets or plates in repair and strengthening of concrete structures, and the use of FRP meshes or textiles or fabrics in thin cement products.

The cost of repair and rehabilitation of a structure is always, in relative terms, substantially higher than the cost of the initial structure. Repair generally requires a relatively small volume of repair materials but a relatively high commitment in labor. Moreover the cost of labor in developed countries is so high that the cost of material becomes secondary. Thus the highest the performance and durability of the repair material is, the more cost-effective is the repair. This implies that material cost is not really an issue in repair and that the fact that FRP repair materials are costly is not a constraining drawback [6].

When considering only energy and material resources it appears, on the surface, the argument for FRP composites in a sustainable built environment is questionable. However, such a conclusion needs to be evaluated in terms of potential advantages present in use of FRP composites related to considerations such as:

- Higher strength
- Lighter weight
- Higher performance
- Longer lasting
- Rehabilitating existing structures and extending their life
- Seismic upgrades
- Defense systems
- Space systems
- Ocean environments

In the case of FRP composites, environmental concerns appear to be a barrier to its feasibility as a sustainable material especially when considering fossil fuel depletion, air pollution, Smog, and acidification associated with its production. In addition, the ability to recycle FRP

Composites are limited and, unlike steel and timber, structural components cannot be reused to perform a similar function in another structure. However, evaluating the environmental impact of FRP composites in infrastructure applications, specifically through life cycle analysis, may reveal direct and indirect benefits that are more competitive than conventional materials.

Composite materials have developed greatly since they were first introduced. However, before composite materials can be used as an alternative to conventional materials as part of a sustainable environment a number of needs remain are:

- Availability of standardized durability characterization data for FRP composite materials.
- Integration of durability data and methods for service life prediction of structural members utilizing FRP composites.
- Development of methods and techniques for materials selection based on life cycle assessments of structural components and systems.

Ultimately, in order for composites to truly be considered a viable alternative, they must be structurally and economically feasible. Numerous studies regarding the structural feasibility of composite materials are widely available in literature [7].

However, limited studies are available on the economic and environmental feasibility of these materials from the perspective of a life cycle approach, since short term data is available or only economic costs are considered in the comparison. Additionally, the long term affects of using composite materials needs to be determined. The byproducts of the production, the sustainability of the constituent materials, and the potential to recycle composite materials needs to be assessed in order to determine of composite materials can be part of a sustainable environment. Therefore in this chapter describe the physicochemical properties of polymers and composites more used in Civil Engineering. The theme will be addressed in a simple and basic for better understanding.

2.2 Material Behavior

2.2.1 General

Reinforced concrete externally bonded with FRP consists of four major components: Concrete, reinforcing steel, FRP and adhesive. This implies a highly nonlinear analysis challenge that involves complications such as extensive cracking and local effects.

A general approach to model such a problem is to select a suitable numerical approach to treat the response of each component separately and then obtain their combined effects by imposing the condition of material continuity. Thus, a complete analysis includes selecting a suitable numerical method, modeling each material using appropriate laws, and modeling the interaction between the materials. Therefore, the properties of each material should be known. This section provides information on concrete, steel, FRP materials and bond between FRP and concrete.

2.2.2 Concrete

Concrete is the most widely used construction material in the world. Concrete is composed of aggregate, cement, water and admixture. Concrete is generally weak in tension and strong in compression. This is due to the fact that the aggregate-mortar interface has lower strength than mortar and this yield to weak concrete under tension [8].

The stress-strain relationship for concrete under compression is initially linear elastic until micro-crack initiation. After that, the behaviour becomes nonlinear. After the ultimate compressive strength the stress decreases with increasing strain, See Figure 3a [9]. Under uni-axial tension the stress–strain response follows a linear elastic relationship until the value of the failure stress is reached. The failure stress corresponds to the onset of micro cracking in the concrete material. Beyond the failure stress the formation of micro-cracks is represented with a softening stress–strain response, Figure 3b. The softening curve of concrete under tension could be represented as follows; see Figure 4 [10], where f_{ct} is the tensile strength and G_f is the fracture energy of concrete.

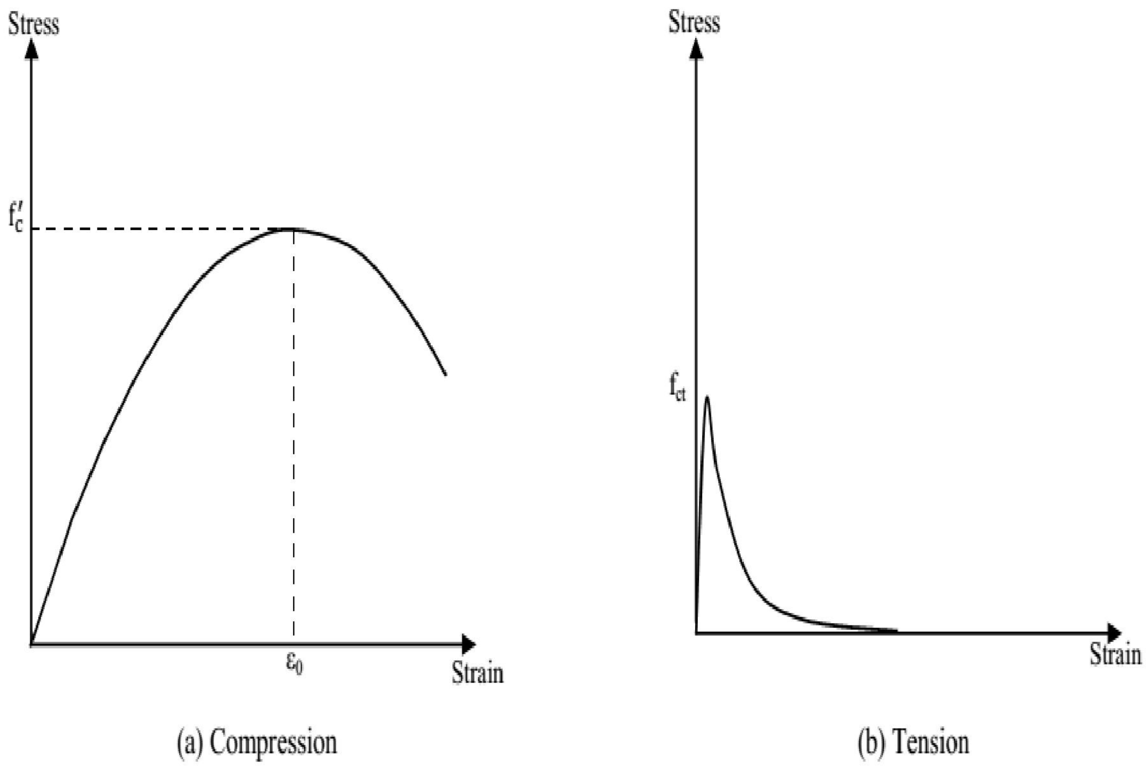


Figure 3Uni-axial stress-strain curves of concrete.

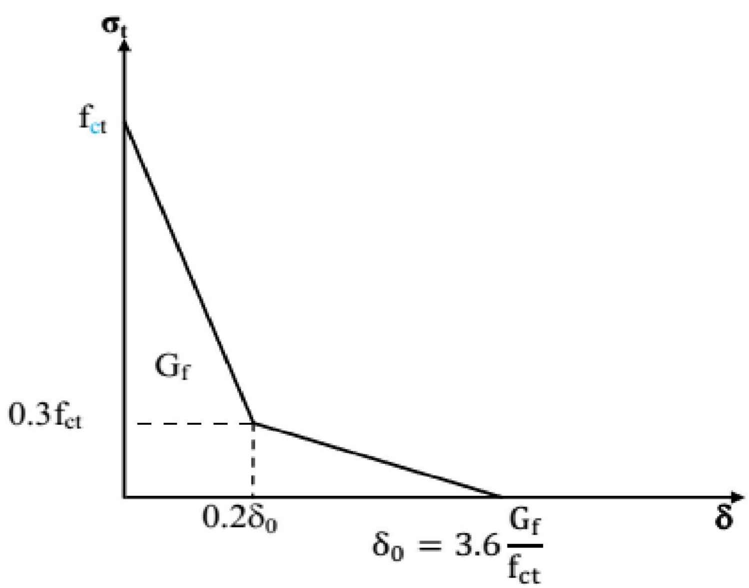


Figure 4 Softening curve of concrete under uni-axial tension, [10]

There are several frameworks of mechanics that can characterize concrete behaviour. The level of complexity of the model is directly related to the ability of the model to capture important features of mechanical behaviour of concrete such as softening. Several models are available for that.

One of the models is the discrete crack model. In this approach, the cracks are defined along element boundaries. The response of concrete in compression could be modeled by Drucker-Prager perfect plasticity, [11].

Another model is the smeared crack model. In the smeared cracking approach, cracking of the concrete occurs when the principal tensile stress exceeds the tensile strength. The elastic modulus of the material is then assumed to be zero in the direction parallel to the principal tensile stress direction, [12] and [13].

The third approach is a plastic damage model. Plastic damage models have been used successfully for predicting the response of standard concrete tests in both tension and compression. The nonlinear material behaviour of concrete can be attributed to two distinct material mechanical processes; plasticity and damage mechanisms. Hardening variables are used to represent the damage in concrete. Stiffness degradation is evaluated to represent the uni-axial tensile and compressive stress-strain response. This model assumes that the main two failure mechanisms are tensile cracking and compressive crushing of the concrete material, [14] and [15].

2.2.3 Reinforcing Steel

Figure 5 shows a typical stress-strain relationship for reinforcing steel. Steel is initially linear-elastic for stress less than the initial yield stress. At ultimate tensile strain, the reinforcement begins to neck and strength is reduced. At a maximum strain, the steel reinforcement fractures and load capacity is lost, ASTM A615 [16]. This steel response may be defined by a few material parameters as identified in Figure 5. These include the elastic modulus, Es, the yield strength, fy, the strain at which peak strength is achieved, ε_u, the peak strength, fu, the strain at which fracture occurs, ε_{max}, and the capacity prior to steel fracture, fs.

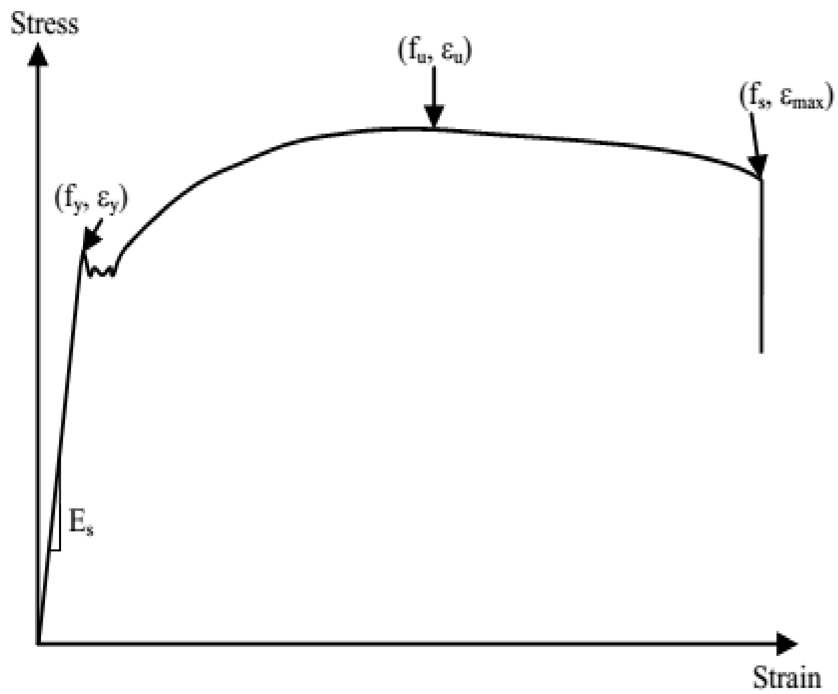


Figure 5 Tensile stress-strain curve for typical reinforcing steel bar

For general engineering applications, an elastic-plastic constitutive relationship, either with or without strain hardening, is normally assumed for ductile reinforcing steel, as shown in Figure 6. In an elastic hardening model it is assumed that steel shows some hardening after it yields, [13]. An elastic-perfectly plastic model generally yields acceptable results for the response prediction of RC members, [17].

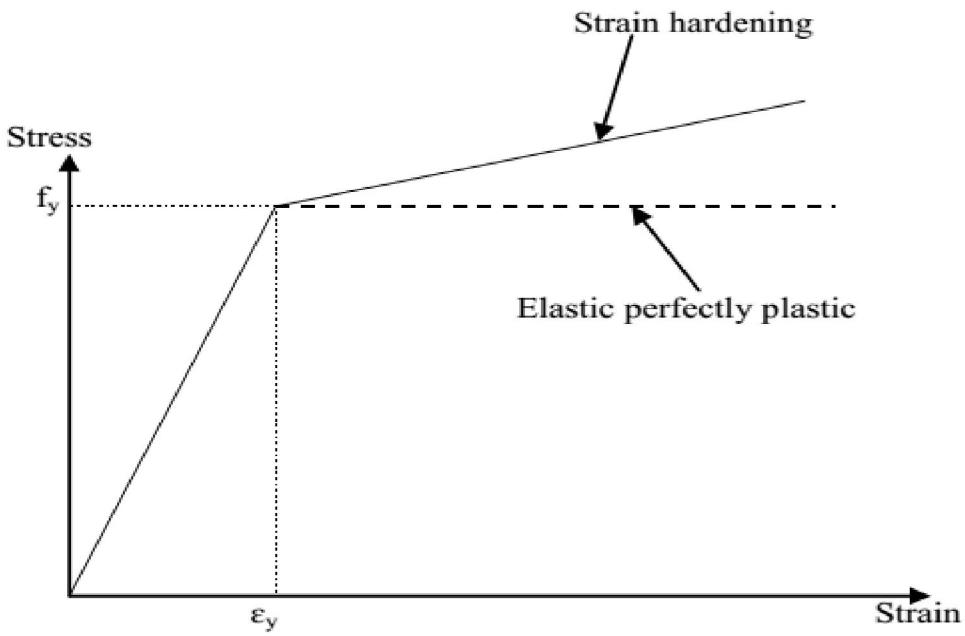


Figure 6 Idealized stress-strain curve for reinforcing steel

2.2.4 FRP Material

Fibre reinforced polymer composites consist of high strength fibres embedded in a matrix of polymer resin as shown in Figure 7.

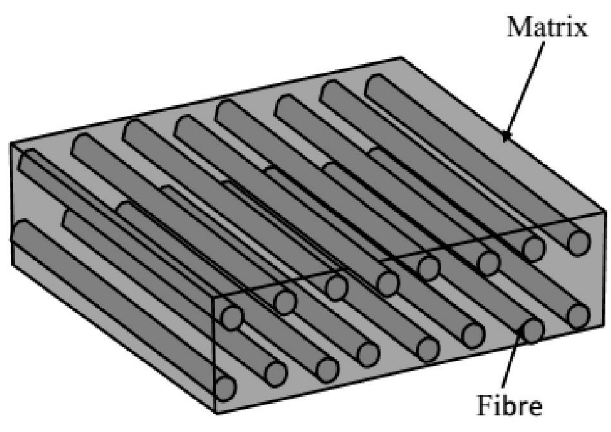


Figure 7 A schematic diagram showing a typical unidirectional FRP plate

In FRP, the fibres provide both load carrying capacity and stiffness to the composite while the matrix is to ensure sharing of the load among fibres and to protect the fibres themselves from the environment. Typical properties for epoxy are given in Table 1. Most FRP materials are made of fibres with high strength and stiffness, while their strain at failure is lower than that of the matrix.

Fibres typically used in FRP are glass, carbon and aramid. Typical values for properties of the fibres are given in Table 1. Carbon fibres are the stiffest, most durable and most expensive fibres. Carbon is quite resistant to most environmental impact. Glass fibres have lower strength and significantly lower stiffness but also a lower cost. Unprotected glass fibres degrade in most environments. Finally, aramid fibres have mechanical characteristics between those of glass and carbon, [18].

The fibre behavior is linear elastic up to failure, with no significant yielding compared to steel. Figure 8 shows the stress-strain relationship for fibre, matrix and the resulting FRP material. Before the yielding of the matrix, the strain in fibre and matrix is the same. After the yielding of the matrix, a knee will appear in the stress-strain curve due to the fact that the matrix no longer contributes to the stiffness.

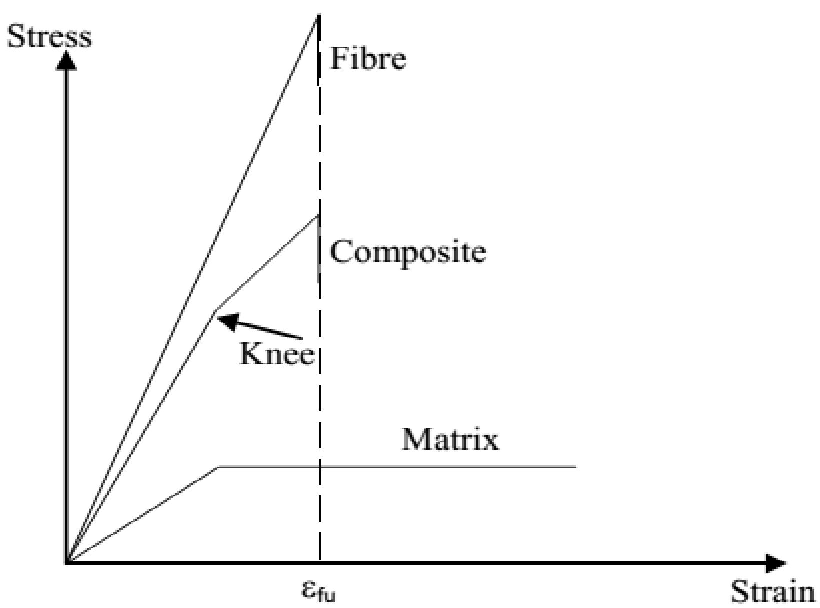


Figure 8 Stress-strain curves for typical fibre, resin and FRP composite, [18]

The mechanical properties of composites are dependent on the fibre properties, matrix properties, fibre-matrix bond properties, fibre amount and fibre orientation distribution. A composite with all fibres in one direction is designated as unidirectional. If the fibres are woven, or oriented in many directions, the composite is bi- or multidirectional.

Since it is mainly the fibres that provide stiffness and strength composites are often anisotropic with high stiffness in the fibre direction(s). In strengthening applications, unidirectional composites are predominantly used, Figure 8. The approximate stiffness and strength of a unidirectional FRP with a 65% volume fraction of carbon fibre is given in Table 1. As a comparison the corresponding properties for steel are also given.

Table 1Typical strength and stiffness values for materials used in retrofitting, [18].

Material	Tensile strength (MPa)	Modulus of elasticity (GPa)	Density (kg/m^3)	Modulus of elasticity to density ratio (Mm^2/s^2)
Carbon	2200-5600	240-830	1800-2200	130-380
Aramid	2400-3600	130-160	1400-1500	90-110
Glass	3400-4800	70-90	2200-2500	31-33
Epoxy	60	2.5	1100-1400	1.8-2.3
CFRP	1500-3700	160-540	1400-1700	110-320
Steel	280-1900	190-210	7900	24-27

An isotropic linear elastic model is usually used to model FRP plate behaviour, if the direction of fibres is parallel to that of the principal stresses, [19]. Orthotropic linear elastic behaviour can be taken into consideration since the FRP material essentially has orthotropic behavior, [20].

2.2.5 Bond between FRP and Concrete

Adhesives are used to attach the composites to other surfaces such as concrete. The most common adhesives are acrylics, epoxies and urethanes. Epoxies provide high bond strength with high temperature resistance, whereas acrylics provide moderate temperature resistance with good strength and rapid curing. Several considerations are involved in applying adhesives effectively.

Careful surface preparation such as removing the cement paste, grinding the surface by using a disc sander, removing the dust generated by surface grinding using an air blower and careful curing are critical to bond performance, [21].

Often the most critical part of FRP application is the adhesive layer between the composite material and substrate, Figure 9.

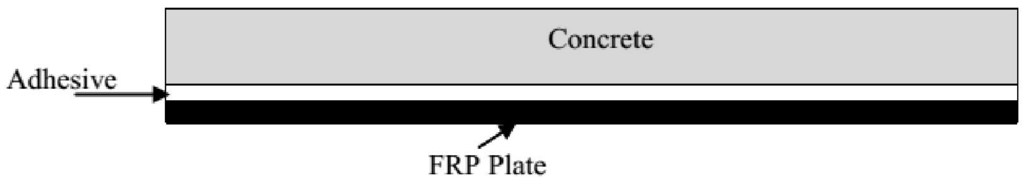


Figure 9 Concrete-FRP system

Bond failure implies complete loss of composite action between concrete and laminate. This type of failure appears in the region of the bond as one of several types of cracking or separation patterns. Materials de-cohesion and interface failure modes may occur. Material de-cohesion includes FRP delamination, cohesive failure of the adhesive and concrete substrate fracture. Interface failures are concrete-steel separation, FRP-adhesive separation and concrete-adhesive separation, which are considered to be dominating in retrofitting in most studies. Figure 10 illustrates each of the possible debonding modes.

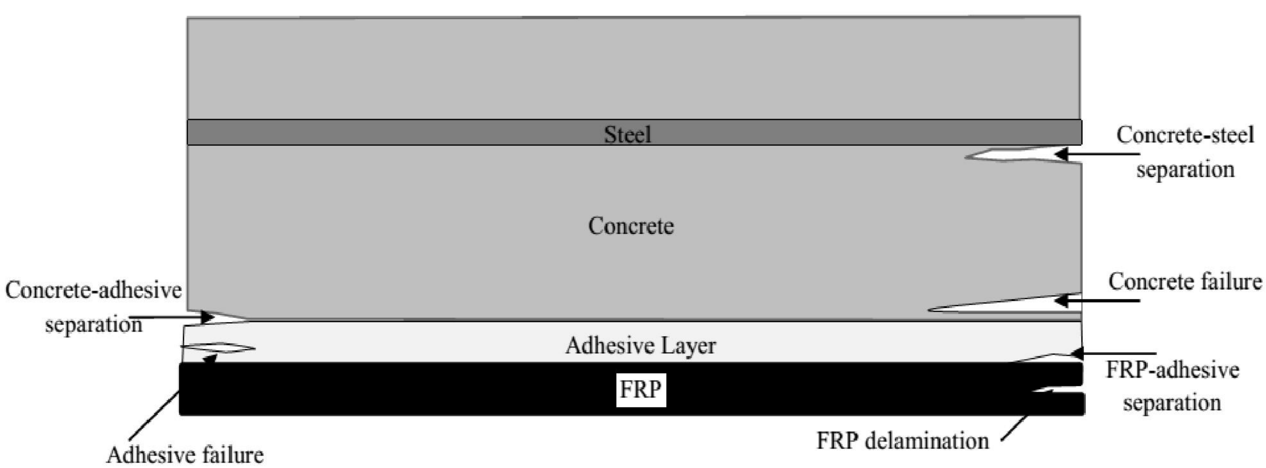


Figure 10 Failure modes at concrete plate bond

The FRP-concrete interface is usually considered as a perfect bond, [22]. However, it is important to consider the compliance of the bond between concrete and FRP since most research indicate that debonding is the dominating type of failure. Only few studies were focused on the bond, [17], and [19].

The bond behaviour could be considered by using a bond-slip model to represent the behaviour of the interface layer. In most studies the bond-slip curve developed was based on axial strain measurement.

The linear-brittle model was developed by Neubauer and Rostasy [23], Figure 11. This bond-slip model does not consider the softening behaviour. Therefore, the ultimate load computed using this bond-slip model is the load corresponding to initiation of interfacial micro-cracking, which in reality can be lower than the bond failure load.

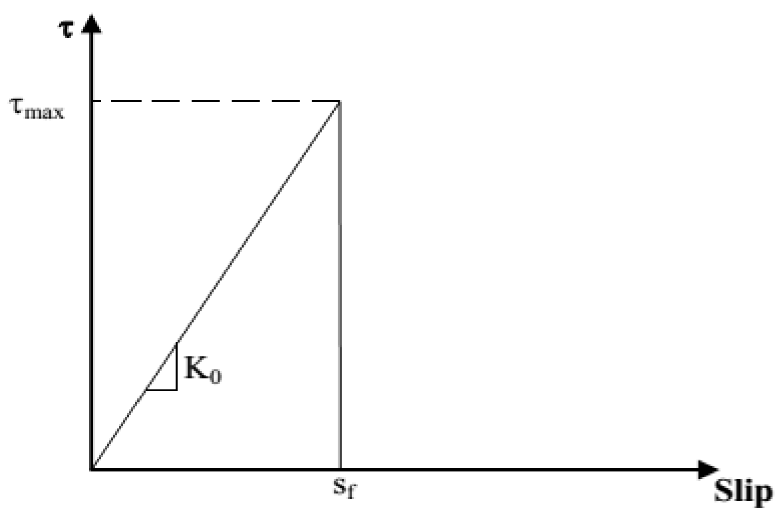


Figure 11 Bond-slip curve, Neubauer and Rostasy [23]

Bond slip should be represented by an ascending and a descending branch, [24] and [25], Figure 12. A bilinear bond-slip curve was proposed by Monti[26]. This model is linear up to maximum bond stress and then the stress decreases to zero stress at ultimate slip. The area underneath the stress-slip curve represents the fracture energy value.

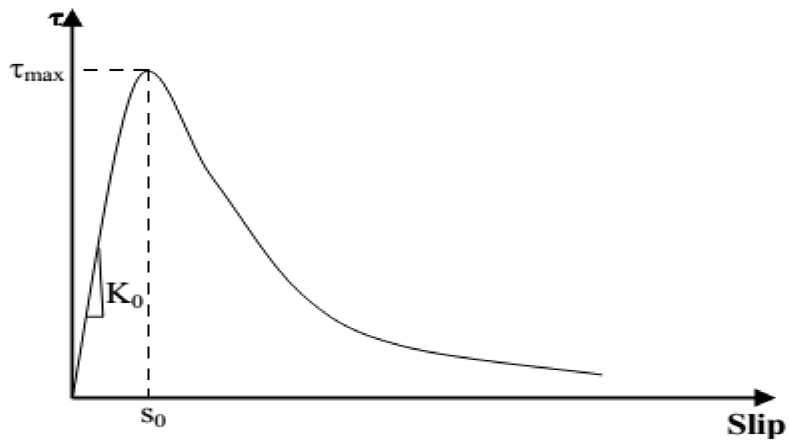


Figure 12- Bond-slip curve, According to Nakaba [24] and Savioa [25]

Lu simulated the behaviour of specimen by FEM. Bond stress and fracture [27] energy parameters were obtained by fitting the simulation results of strain distribution along the FRP and ultimate load with experimental results. The bond-slip curves proposed by Lu. [27] are the precise model, simplified model and the bilinear model, Figures 13-15.

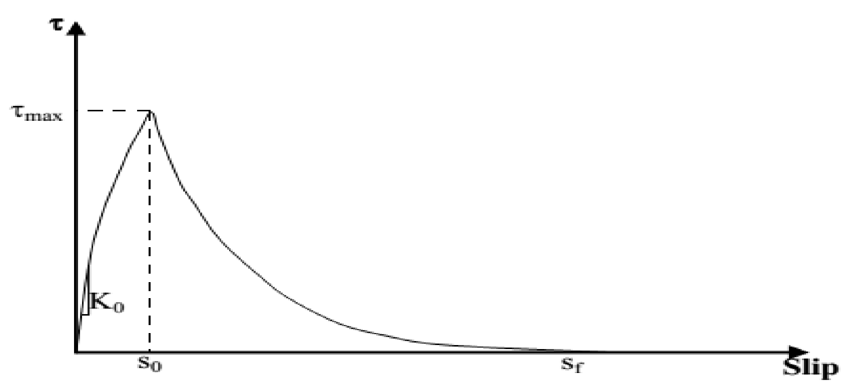


Figure 13 Bond-slip curve, precise model, according to Lu [27]

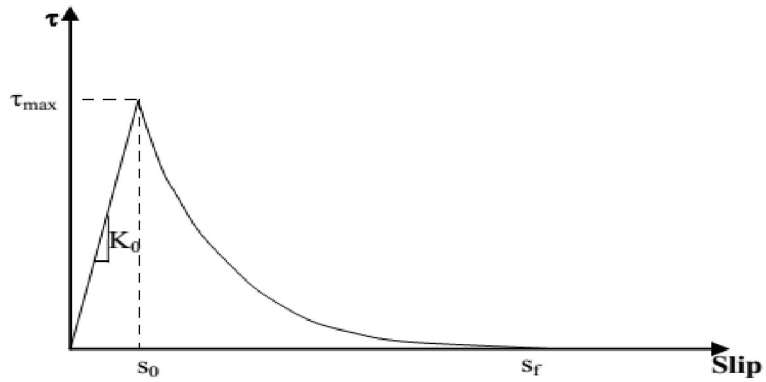


Figure 14 Bond-slip curve, simplified model, according to Lu. [39]

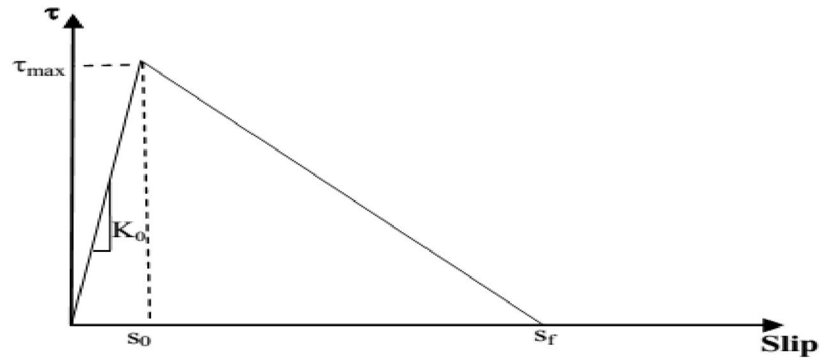


Figure 15 Bond-slip curve, bilinear model, according to Lu [27]

Much research was focused on finding the bond strength and fracture energy. Neubauer and Rostasy [28] developed a relation to describe the bond strength and fracture energy expressed as a function of tensile strength of concrete. Brosens and Van Gemert [29], Nakaba. [24] and Lu [27] developed a relation to describe the bond strength and the fracture energy expressed as a function of tensile strength of concrete and the ratio between the plate and the concrete width. Ulaga [30] developed a relation expressed as a function of the compressive strength of concrete. Based on different types of interfacial bond stress-slip relationships, Yuan [31] proved that the maximum interfacial bond force can be expressed as a function of fracture energy and FRP stiffness.

Even though extensive work has been done on the use of FRP laminates in retrofitting there is a need for further refinement of models and further parameter studies. From the literature review, it can be concluded that the interface zone has been modelled with linear or with nonlinear models.

Researchers have reported on different failure modes. It is important to understand under what circumstances a certain failure mode will occur. Despite that many models have been developed to represent the bond strength and fracture energy, it is necessary to develop a model that depends on the adhesive properties, because these properties play a significant role in debonding failure beside concrete properties.

Main weaknesses in the available guidelines are lack of a unified design approach especially in the design rules concerning composite action. Better understanding is needed and development of simple design models for mechanisms associated with debonding is an important task.

2.3 FRP, applications

Fiber-reinforced plastics are best suited for any design program that demands weight savings, precision engineering, finite tolerances, and the simplification of parts in both production and operation. A molded polymer artifact is cheaper, faster, and easier to manufacture than cast aluminum or steel artifact, and maintains similar and sometimes better tolerances and material strengths. The Mitsubishi Lancer Evolution IV also used FRP for its spoiler material [32-34].

2.3.1 Structural applications of FRP

FRP can be applied to strengthen the beams, columns and slabs in buildings. It is possible to increase strength of these structural members even after these have been severely damaged due to loading conditions. For strengthening beams, two techniques are adopted. First one is to paste FRP plates to the bottom (generally the tension face) of a beam. This increases the strength of beam, deflection capacity of beam and stiffness.

Alternatively, FRP strips can be pasted in 'U' shape around the sides and bottom of a beam, resulting in higher shear resistance. Columns in building can be wrapped with FRP for achieving higher strength. This is called wrapping of columns. The technique works by restraining the lateral expansion of the column. Slabs may be strengthened by pasting FRP strips at their bottom (tension face). This will result in better performance, since the tensile resistance of slabs is supplemented by the tensile strength of FRP. In the case of beams and slabs, the effectiveness of FRP strengthening depends on the performance of the resin chosen for bonding [34].

For structural applications, FRP is mainly used in two areas. The first area involves the use of FRP bars instead of steel reinforcing bars or pre-stressing strands in concrete structures. The other application, which is the focus of this thesis, is to strengthen structurally deficient structural members with external application of FRP.

Retrofitting with adhesive bonded FRP has been established around the world as an effective method applicable to many types of concrete structural elements such as columns, beams, slabs and walls.

As an example, a highway RC bridge slab in China was retrofitted using CFRP as shown in Figure 2(a) and a column in India was retrofitted using glass FRP wrapping as shown in Figure 16(b),[35].

FRP plates can be bonded to reinforced concrete structural elements using various techniques such as external bonding, wrapping and near surface mounting. Retrofitting with externally bonded FRP has been shown to be applicable to many types of RC structural elements. FRP plates or sheets may be glued to the tension side of a structural member to provide flexural strength or glued to the web side of a beam to provide shear strength. FRP sheets can also be wrapped around a beam to provide shear strength and be wrapped around a column to provide confinement and thus increase the strength and ductility. Near surface mounting consists of sawing a longitudinal groove in a concrete member, applying a bonding material in the groove and inserting an FRP bar or strip.

(a) Flexural strengthening of a highway RC bridge slab in China.

(b) Seismic retrofit of supporting columns for a cryogenic tank in Gujarat, India

Figure 16 Examples of use of FRP in existing structures, [35].

2.4 Review on Experimental works

Investigation of the behavior of FRP retrofitted reinforced concrete structures has in the last decade become a very important research field. In terms of experimental application several studies were performed to study the behavior of retrofitted beams and how various parameters influence the behavior. The effect of number of layers of CFRP on the behavior of a strengthened RC beam was investigated by Toutanji [36], tested simply supported beams with different numbers of CFRP layers. The specimens were subjected to a four-point bending test. The results showed that the load carrying capacity increases with an increased number of layers of carbon fiber sheets.

Investigation of the effect of internal reinforcement ratio on the behavior of strengthened beams has been performed by Esfahani [37]. Specimens with different internal steel ratio were strengthened in flexure by CFRP sheets. The authors reported that the flexural strength and stiffness of the strengthened beams increased compared to the control specimens. With a large reinforcing ratio, they also found that failure of the strengthened beams occurred in either interfacial debonding induced by a flexural shear crack or interfacial debonding induced by a flexural crack.

A test program on retrofitted beams with shear deficiencies was done by Khalifa. [38]. the experimental results indicated that the contribution of externally bonded CFRP to the shear capacity of continuous RC beams is significant.

There are three main categories of failure in concrete structures retrofitted with FRP that have been observed experimentally, [37-41]. The first and second type consist of failure modes where the composite action between concrete and FRP is maintained. Typically, in the first failure mode, the steel reinforcement yields, followed by rupture of CFRP as shown in Figure 17(a). In the second type there is failure in the concrete. This type occurs either due to crushing of concrete before or after yielding of tensile steel without any damage to the FRP laminate, Figure 17(b), or due to an inclined shear crack at the end of the plate, Figure 17(c). In the third type, the failure modes involving loss of composite action are included. The most recognized failure modes within this group are debonding modes.

In such a case, the external reinforcement plates no longer contribute to the beam strength, leading to a brittle failure if no stress redistribution from the laminate to the interior steel reinforcement occurs. Figures 17(d)-(g) show failure modes of the third type for RC beams retrofitted with FRP. In Figure 17(d), the failure starts at the end of the plate due to the stress concentration and ends up with debonding propagation inwards. Stresses at this location are essentially shear stress but due to small but non-zero bending stiffness of the laminate, normal stress can arise. For the case in Figure 17(e) the entire concrete cover is separated. This failure mode usually results from the formation of a crack at or near the end of the plate, due to the interfacial shear and normal stress concentrations.

Once a crack occurs in the concrete near the plate end, the crack will propagate to the level of tensile reinforcement and extend horizontally along the bottom of the tension steel reinforcement. With increasing external load, the horizontal crack may propagate to cause the concrete cover to separate with the FRP plate. In Figures 2.16(f) and (g) the failure is caused by crack propagation in the concrete parallel to the bonded plate and adjacent to the adhesive to concrete interface, starting from the critically stressed portions towards one of the ends of the plate. It is believed to be the result of high interfacial shear and normal stresses concentrated at a crack along the beam. Also mid span debonding may take concrete cover with it.

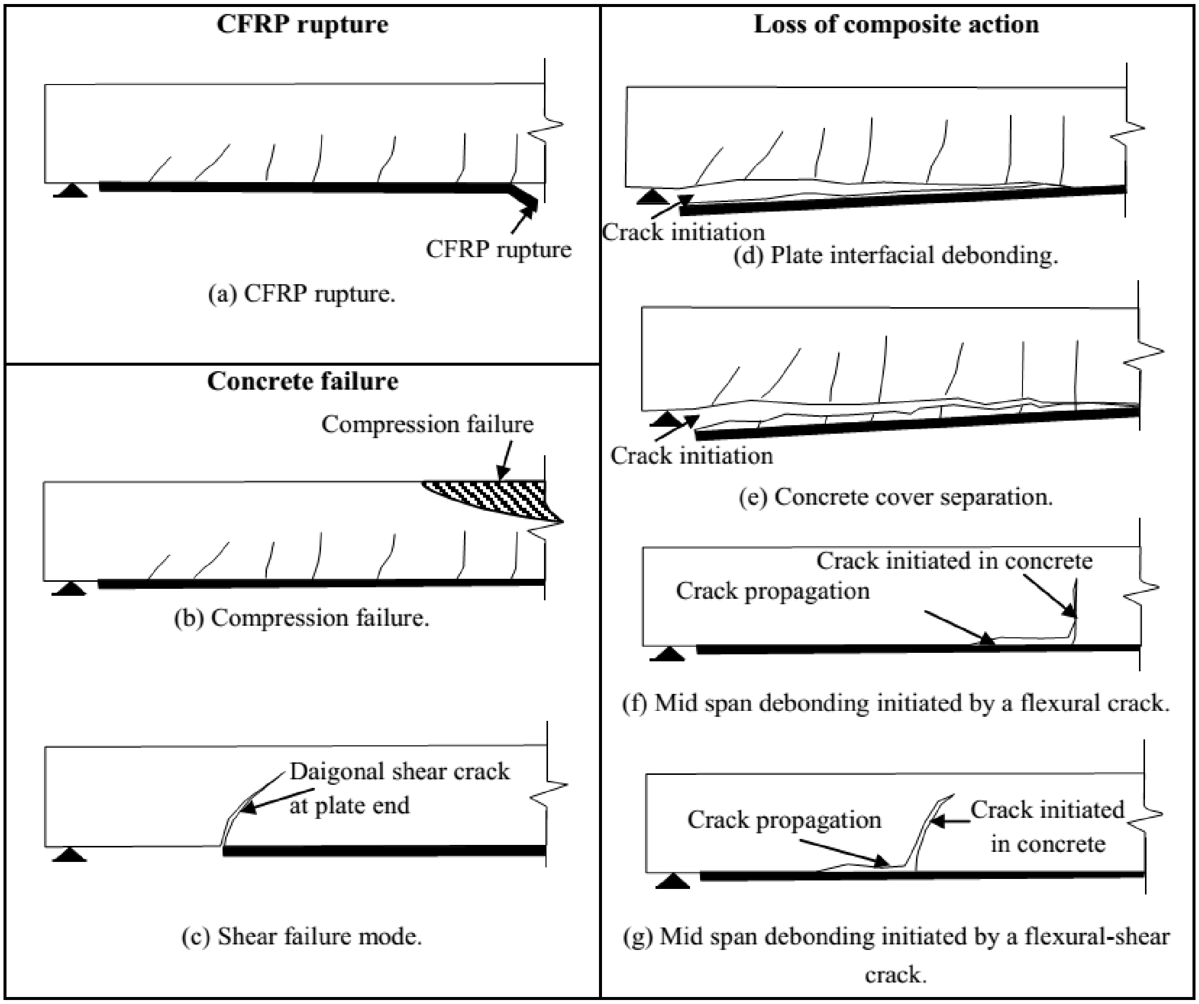


Figure 17 Failure modes in beam retrofitted in flexure [37].

2.5 Problems of strengthening RC beams

The study indicates that researchers have used FRP plane laminates at different number of layers. According to ACI code assumptions, the bond between laminates and concrete surface is perfect (ACI 440.2R-08). But most of the cases discussed in failure modes of a strengthened beam, the failure of beams occurred due to debonding of laminates from concrete surface. In those cases, the beams failed by premature failure which means, beams failed under the initial load.

Also it has been noticed that, failure due to debonding of laminates occurs for beams retrofitted only at the bottom. From theoretical analysis, the researchers have used only one tool for model development such as either finite element analysis or neural network.

2.6 Proposed method of strengthening the RC beam

To overcome the problem discussed in this paper, in the proposed work, FRP composites are used to develop a new profile and investigations for its physical dimensions and structural behaviors, in flexural members. FRP laminates will be provided with full length of the beam to take into account shear and bending. To avoid premature failure, FRP laminates are provided at bottom and are extended to the sides also. Most of the authors have used epoxy resins for attaching FRP laminates with concrete surface due to its superior property by comparing other adhesives. The same resins will be used for proposed work. Thirteen authors out of the reviewed literatures have used two point load system for their experimental setup.

2.7 Review on modeling works

Many models currently exist for reinforced concrete retrofitted with FRP. Several different approaches have been considered. Some models use simple material models and are restricted to 2D and others use nonlinear elasticity or plasticity models to capture the more complicated effects and predict the behaviour of retrofitted reinforced concrete in a general sense. Each approach has its strengths, complexity level, and complications. A 2D model was developed by Supaviriyakit [42] for analyses of RC beams strengthened with externally bonded FRP plates.

The RC element considered the effect of crack and reinforcing steel as being smeared over the entire element. Perfect compatibility between cracked concrete and reinforcing steel was assumed. The FRP plate was modelled as an elastic brittle element. As the epoxy is usually stronger than the concrete, perfect bond between FRP and concrete was assumed.

The orthotropic properties of FRP were taken into consideration by Hu [43] in modelling the behaviour of a retrofitted beam. They assumed perfect bond between the FRP plate and concrete.

The effect of anchorage length of near surface mounted reinforcement (NSMR) was studied by Lundqvist. [44], conducted numerical analyses of three different FRP strengthening techniques to find a critical anchorage length, where a longer anchorage length does not contribute to the load bearing capacity. They assumed perfect bond between the plate and concrete. The results showed that a critical anchorage length exists for plates and sheets as well as for NSMR. Bond is a critical parameter in strengthening systems as it provides the shear transfer between concrete and FRP necessary for composite action. Lim. [45] presented a numerical model to simulate the interface fracture behaviour of concrete strengthened with external composite plates. They adapted the fictitious crack model, [46] with a nonlinear fracture mechanics concept to describe the constitutive relationship at the element level. They found that the interface material properties had significant influence on the interface stress distributions. Furthermore, Camata [47], investigated RC members strengthened in flexure by FRP plates. The model considers the actual crack patterns observed in the test using a smeared and interface crack model. The results show that debonding and concrete cover splitting failure mode always occur by crack propagation inside the concrete. A FE analysis was performed by Neale [48], to simulate the nonlinear behaviour of shear strengthened beams and two-way slabs. A plasticity–based concrete constitutive model was used. An elastic–plastic response was assumed for the steel and the FRP was modelled as linear elastic until failure. A bond slip model was incorporated to the analysis to simulate the FRP concrete interface.

3. Experimental Program

3.1 Introduction

This chapter reports the experimental program of the research that was conducted. The experimental work will be described in following sections: Section 3.2 provides a description of the experimental work and the material properties of the beam specimens. Section 3.3 describes the experimental study that was done and what was used to strengthen the beams. In Section 3.4 Experimental procedures are shown. Details of the specimens, test set-up and the properties of the materials used are presented in this chapter. The results from these experiments are discussed in chapter 4.

3.2 Material property

An ordinary strength concrete mix was prepared using Ordinary Portland cement with a specific gravity of 3.15 is used.

Fine Aggregate (d < 5mm)

Table 2 Sieve analysis result of fine aggregate

Sieve Size [mm]	Weight Retained [gm]	% Retained	Cummulative Courser [%]	Cummulative Passing [%]
9.50	0.00	0.00	0.00	100.00
4.75	14.00	2.52	2.52	97.48
2.36	65.00	11.70	14.22	85.78
1.18	176.63	31.79	46.01	53.99
0.60	138.00	24.84	70.84	29.16
0.30	67.00	12.06	82.90	17.10
0.15	65.00	11.70	94.60	5.40
Pan	30.00	5.40	100.00	0.00
Sum	**555.6**			

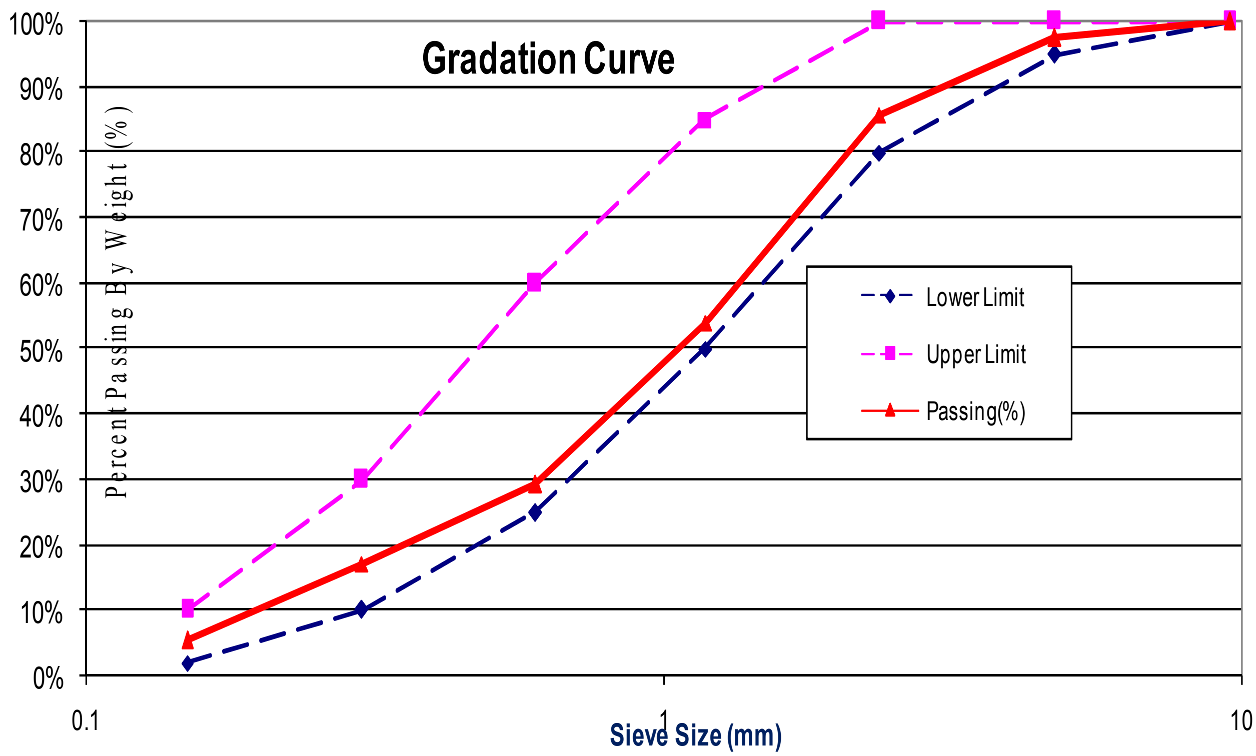


Figure 18 gradation curve of fine aggregate

Summary of fine aggregate can be shown

Table 3 summarized properties of fine aggregate

.

Fineness Modulus=	3.11
Specific Gravity(SSD)=	2.64
Absorbtion Capacity (%)=	2.01
Silt Content (%)=	4.82

Coarse Aggregate (5 mm ≤ d ≤ 20 mm)

Table 4 Sieve analysis result of coarse aggregate

Sieve Size [mm]	Weight Retained [gm]	% Retained	Cummulative Courser [%]	Cummulative Passing [%]
37.50	0.00	0.00	0.00	100.00
19.00	2563.41	65.75	65.75	34.25
9.50	369.00	9.46	75.22	24.78
4.75	703.34	18.04	93.26	6.74
Pan	262.87	6.74	100.00	0.00
Sum	**3898.6**			

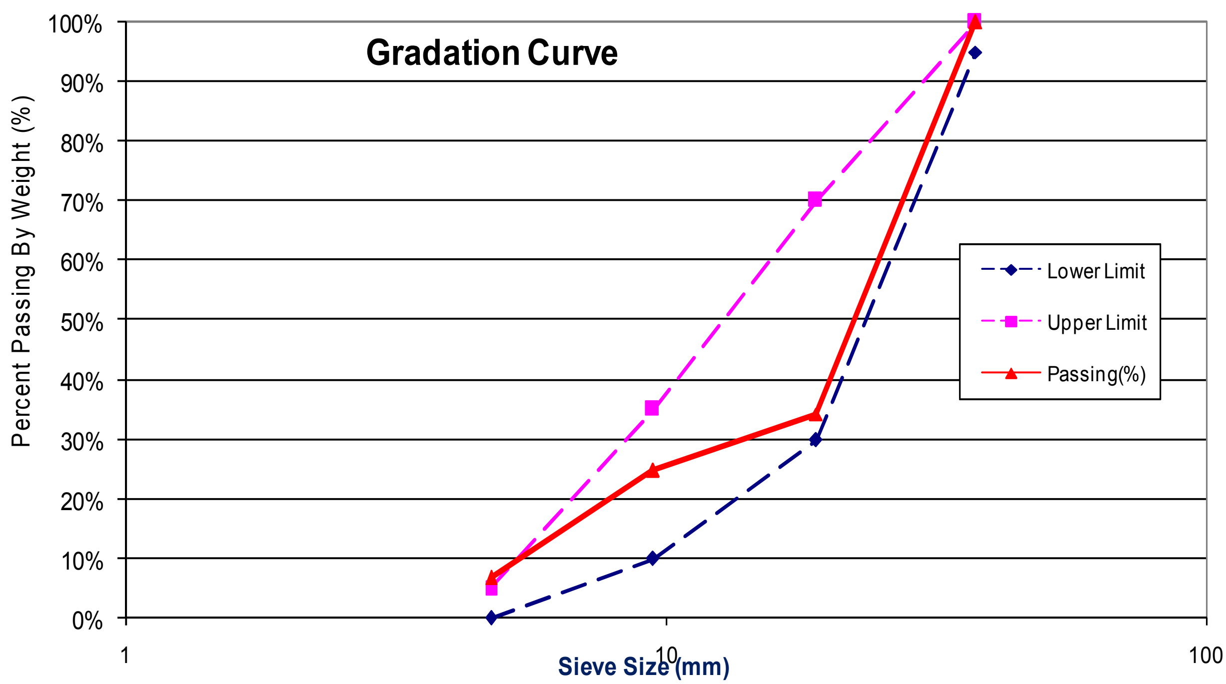


Figure 19 gradation curve of coarse aggregate

Summary of coarse aggregate can be shown

Table 5 summarized properties of coarse aggregate

Specific Gravity(SSD based)=	**2.82**
Absorbtion Capacity=	**2.44**

The steel bars used for longitudinal reinforcement were tested in uniaxial tension. Details of the material properties for the reinforcing steel are given in Table 6

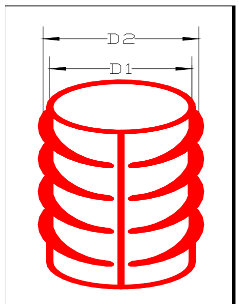


Figure 20 Steel bar detail dimension

Table 6 Material properties of reinforcing steel

Specimen No	Diameter		Yield load [kN]	Yield Stress		Failure Load [kN]	Failure Stress		Elongation [%]	Mass/length kg/m
	D1, [mm]	D2, [mm]		1*, [MPa]	2*, [MPa]		1*, [MPa]	2*, [MPa]		
8-1	7.25	8.12	20.40	494.16	393.94	24.60	595.89	475.04	15.50	0.377
8-2	7.41	8.24	21.60	500.87	405.05	25.40	588.99	476.31	15.50	0.368
8-3	7.36	8.05	20.80	488.90	408.68	24.90	585.27	489.24	15.00	0.376

The GFRP used in this study was supplied by VITRULAN. The laminate has a thickness of 0.69 mm, a width of 20 mm, weight of 168 g/m^2 according to the manufacturer. The plates were supplied in a roll form as shown in Fig. 21.

Figure 21 Roll of GFRP laminate and failure of FRP laminate

The material used for the bonding of GFRP plates to the concrete was an epoxy adhesive with compressive strength 50 MPa and Tensile strength of 18 MPa according to the manufacturer and it was applied with a total thickness equal to 1 mm.The employed epoxy adhesive was Sikadur®332.

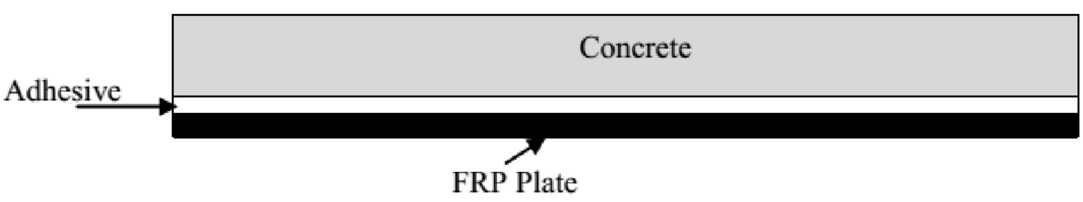


Figure 22 Concrete-FRP system

3.3 Mix design

The concrete mix was designed according to ACI method 211, to have slump of 45 mm and 28 days cylinder compressive strength of 30 MPa. The maximum aggregate size was 10 mm and the free water cement ratio was 0.52. The concrete mix is shown inTable 1

The specific gravity of Fine Aggregates (FA) and Coarse Aggregates (CA) is 2.64 and 2.82 respectively. Ordinary Portland cement was used with specific gravity of 3.15.The mix design is prepared as follows.

Table 7 Mix Proportion

Water	**Cement**	**Fine Agg.**	**Coarse Agg.**
(kg/m^3)	**(kg/m^3)**	**(kg/m^3)**	**(kg/m^3)**
187.20	360.00	716.51	1148.05

The concrete compressive strength test was carried out at day 7, day 21 and day28. Each test consisted of 3 cubes measuring at 150 mm x 150 mm. The test used the MCC computerized control console in the concrete laboratory as shown in Figure 23.

Figure 23 MCC computerized control console for testing concrete tensile strength

The mean compressive strength was determined in compressive tests 28 days after casting of three cubes measuring at 150 mm x 150 mm. The average cylinder compressive strength was found 43.4 MPa. The failure of a specimen is shown Table 8.

The result on the strength development with curing age of concrete are listed in Table 8.

Table 8 Concrete strength development with curing age

Date	Specimen No.	Dimensions [cm]			Area [cm^2]	Failure Load [kN]	Compressive Strength [MPa]
		L	W	H			
7th	1	15.0	15.0	15.0	225.00	564.9	25.1
	2	15.0	15.0	15.0	225.00	570.5	25.4
	3	15.0	15.0	15.0	225.00	580.6	25.8
14th	1	15.0	15.0	15.0	225.00	690.5	30.7
	2	15.0	15.0	15.0	225.00	695.2	30.9
	3	15.0	15.0	15.0	225.00	680.5	30.2
28th	1	15.0	15.0	15.0	225.00	1233.2	54.8
	2	15.0	15.0	15.0	225.00	1235.0	54.9
	3	15.0	15.0	15.0	225.00	1193.5	53.0

3.4 Experimental Procedure

Twelve beams were tested under one point bending after curing. The beams were generally divided into two groups. Controlled beam (BC) and Retrofitted beam (RF). For group RF, focus was on flexural behavior, Total of nine beams was tested. Three different groups of RF were used, with different number of layers GFRP applied. RF1 for one layer, RF2 for two layers and RF3 for three layer of GFRP applied.

Three beams were used as control beams. Finally, the retrofitted beams were loaded until failure and the results were compared with the results of control beams.

3.4.1 Manufacture of beams

The beams had a rectangular cross-section of 150 mm width and 150 mm height, and were 1000 mm long. The beams in group BF were designed to have insufficient flexural strength to obtain a pure flexural failure. They had tension reinforcement of (2ø8), and the steel bars were tied together with 8 mm stirrups c/c 150 mm along the beam. In all the beams, the clear concrete cover to the main flexural reinforcement was set to 25 mm and this cover was expected to avoid splitting bond failure.

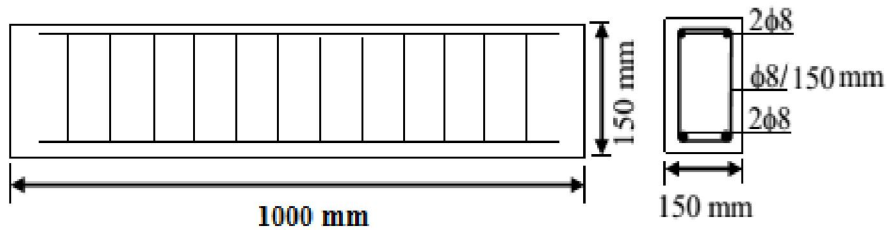


Figure 24 Geometry and reinforcement of beams in groups RF

Figure 25 Produced reinforced concrete beams

3.4.2 Testing of beams

The beams were tested in one point loading. This load forms maximum moment and deflection at center of the beam. The testing equipment was a testing machine of 400 kN capacity jack. A linearly variable differential transducer, LVDT, was used to measure the deflection at midspan.

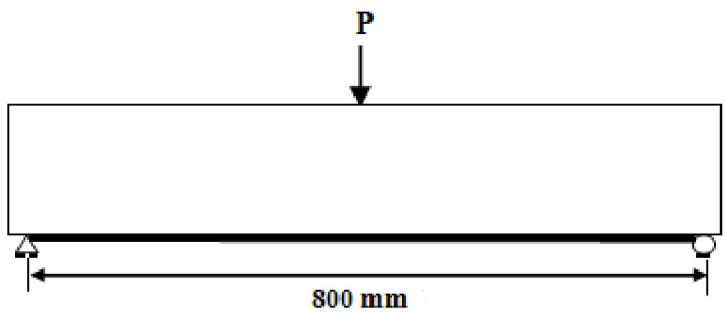


Figure 26 Support and loading position

During the initial set up of the LVDTs, the instruments were calibrated before the test commenced. An automated data acquisition system with a data logger system was used to record the load-deformation from the jacks and the LVDTs.

Figure 27 Test setup

4. Experimental Test Results and Discussions

4.1 Introduction

The test results are presented in this chapter. The behavior of the specimens during the laboratory experiment is discussed. The effects of varying number of layers of FRP on the test beams are elaborated in this chapter.

4.2 Control Beams

The load versus mid span deflection curves for the three control beams are taken. The beams behave in a ductile manner and gives large deflection before the final failure. This is the typical behaviour of an under-reinforced RC member [14]. The difference between the three specimens is rather small, and the mean value, also indicated in the figure, will be used.

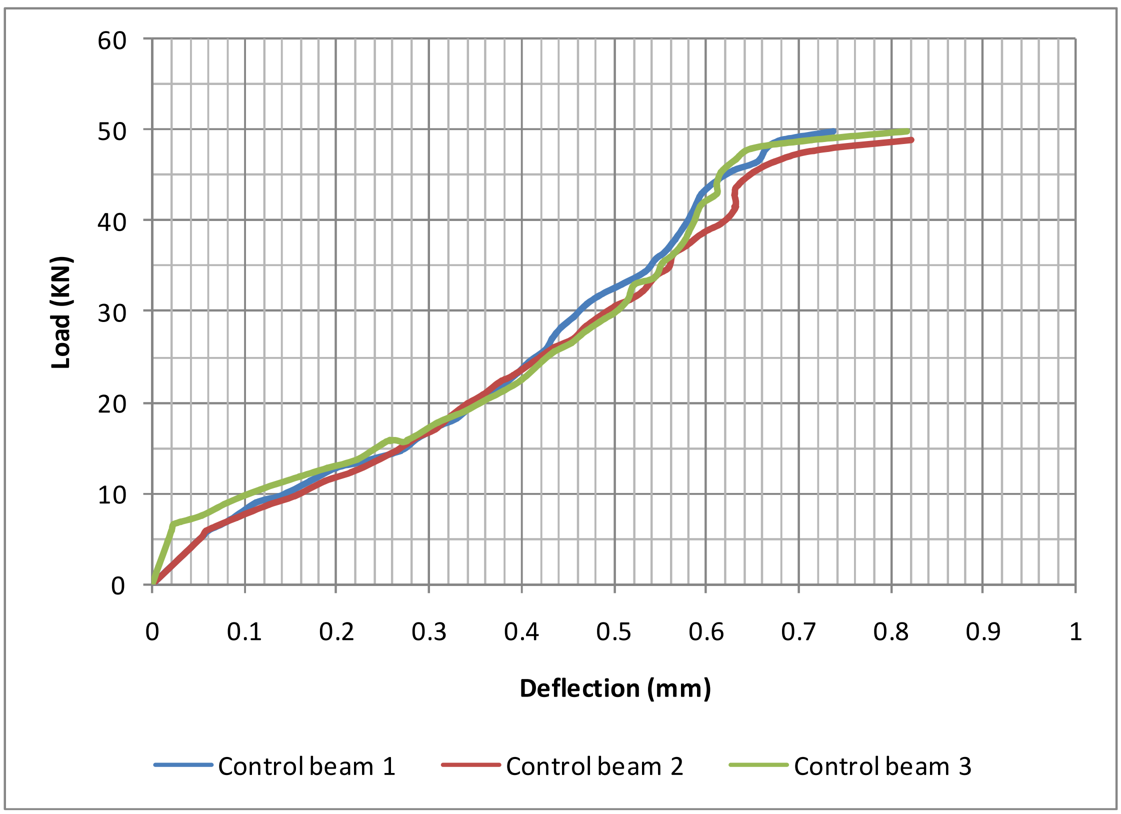


Figure 28 Load Vs deflection of control beams in group BC

The curve includes a linear response up to the load 7.5 kN. The appearance of a crack was first noted at load 40 kN. As the load is increased, flexural cracks increased in number, width and depth. When the load was increased further, flexural failure in tension was the mode failure by yielding of main steel reinforcement. The failure load was recorded when a rapid increasing in the deflection readings occurred obviously which means yielding the main reinforcement in the beam specimens. The mid span deflection curve illustrates the nonlinearities at cracking of the concrete. After 45 kN load flexural cracks formed and widened as loading increased.

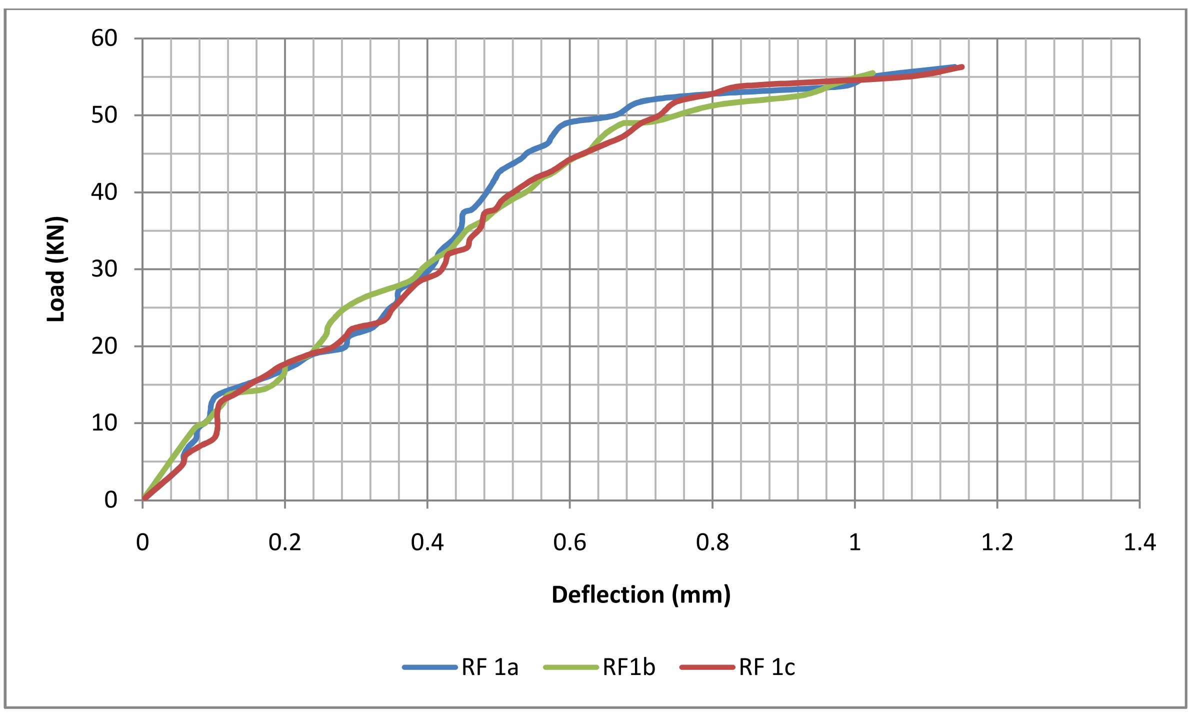
The maximum load was 49.8 kN. After maximum load, the cracks did not grow in length for the remainder of the test but the flexural cracks in the constant moment region widened. The failure of a control beam is shown in Fig 29.

Figure 29 Flexural failures for control beam

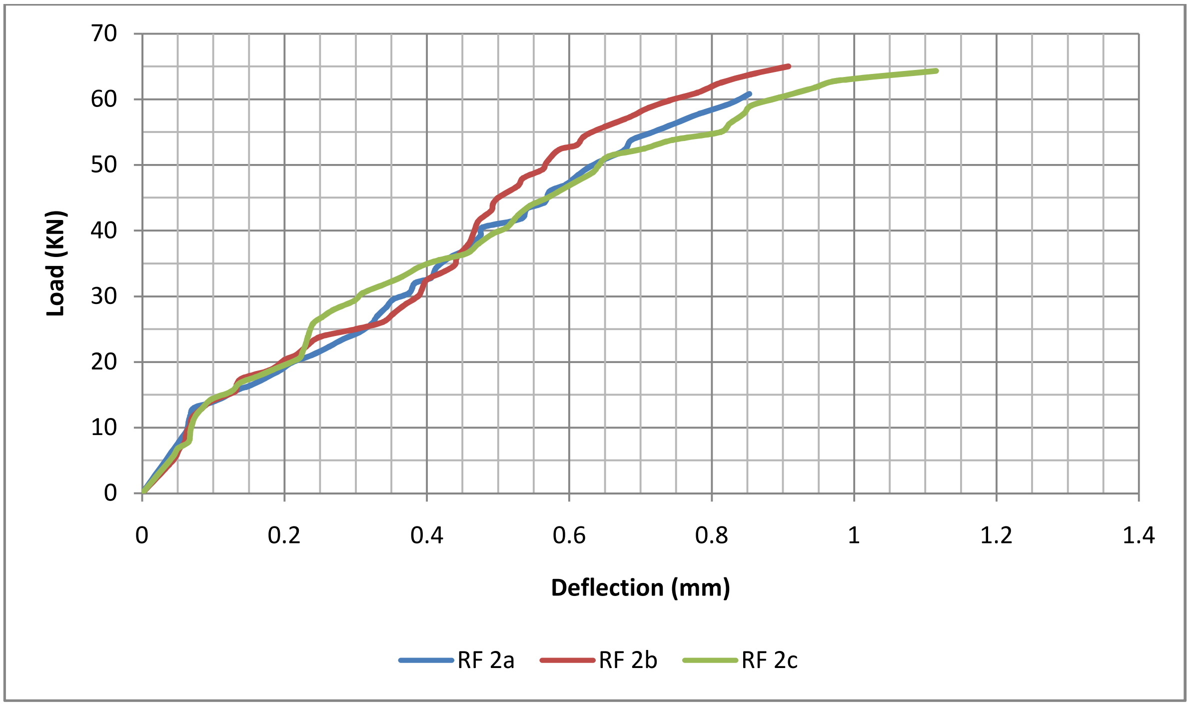
4.3 Retrofitted beams

The load–deflection curves for the individual beams in series RF1, RF2 and RF3 are shown in Fig. 30. The results from the three beams in each series are close, which indicates that the retrofitting was performed in similar manner. The mean value will be used to represent each series.

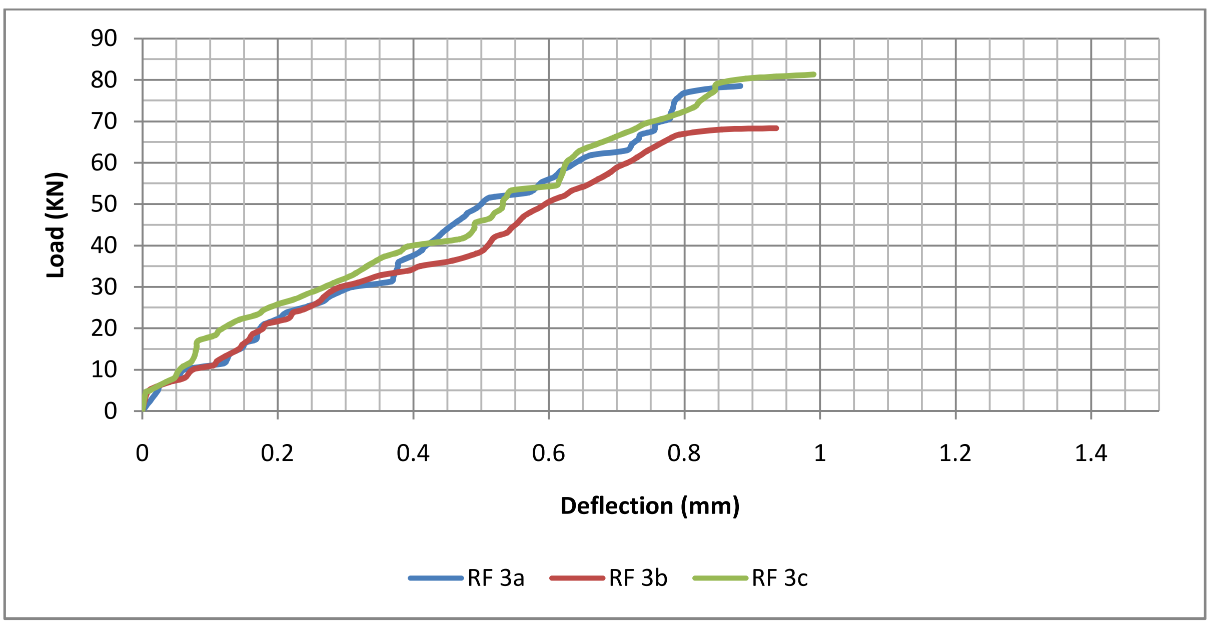
The mean load–deflection curves for the retrofitted beams and for the control beams are shown in Fig. 31. As shown in the figure the stiffness of all beams at small load is almost the same. From cracking stage- the stiffness of the control beam decreases notably due to cracking. The decrease in stiffness is smaller for the retrofitted beams since the GFRP prevents cracks to develop and widen. The increase in number of FRP layers increase in the capacity of beam. This is probably because the layers increment of GFRP strips have an additional capacity to carry moment and are hence more efficient in the cracking zone. Some contribution to the stiffness may also be due to the stiffening of the beam caused by the GFRP outside the cracking region.



(a)



(b)



(c)

Figure 30 Comparison between load-deflection curves for individual retrofitted beams in group RF. (a) series RF1, (b) series RF2 and (c) series RF3

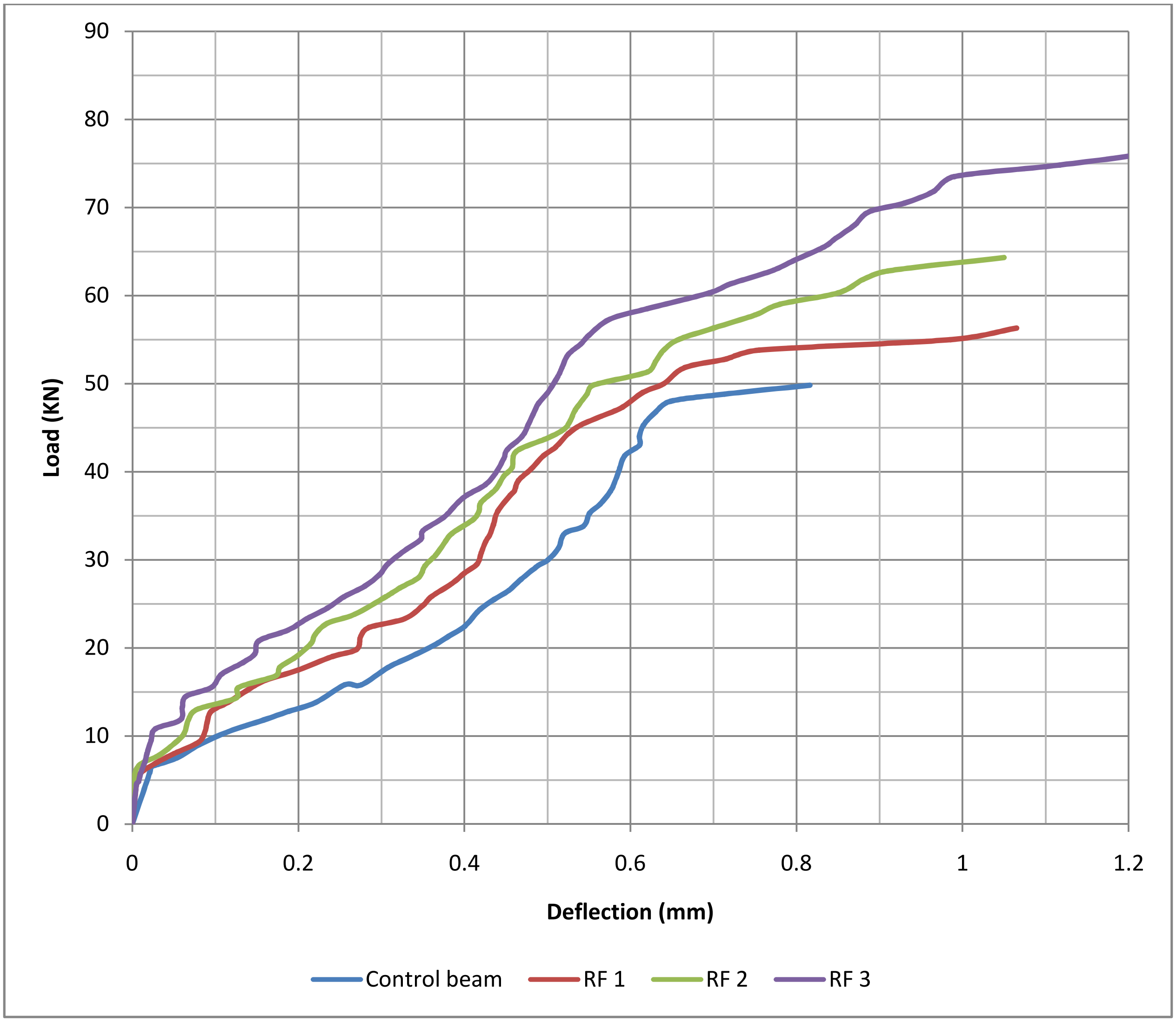


Figure 31 Comparison between load-deflection curves for control beam and RF group beams

In specimen (RF1) is strengthened with one layers of FRP sheet. In this beam, the first observed crack occurred at load about 45 kN in mid support region. As the load was increased, flexural cracks increased in number and width, further flexure-shear cracks appeared at load about 52kN. The failure was rupture of FRP sheet at the middle span when the load reach about 56.3 kN with an increase in ultimate load of about (13.05%) with respect to control beam (BC) as shown in Figure 31.

In specimen (RF2) is strengthened with two layers of FRP sheet. In this beam, the first observed crack at load about 50 kN in center of left span. As the load was increased, flexural cracks increased in number and width, further flexure cracks appeared at load about 56kN.The failure was rupture in FRP sheet at the center of span when the load reach about 64.3 kN with an increase in ultimate load of about (29.11%) with respect to control beam specimen (BC) as shown in Figure 31.

In specimen (RF3) is strengthened with three layers of FRP sheet. In this beam, the first observed crack at load about 64 kN in mid region. As the load was increased flexural cracks increased in number and width, further flexure-shear cracks appeared at load about 70kN. The failure was rupture in FRP sheet at around center of beam when load reach about 78.5kN with an increase in ultimate load of about (57.63%) with respect to control beam (BC) as shown in Figure 31.

The curves reveal that the strengthening process has significantly increased the maximum load in series RF1, RF2 and RF3. The maximum load in series RF1 was 56.3 KN, which is a more than 13.05 % increase compared to the control beam. The maximum load for series RF2 was 64.3 kN, 29.11 % higher than for the control beam. For series RF3 the maximum load was 78.5 kN which corresponds to a 57.63 % increase in maximum load.

Failure modes in retrofitted beam were observed as the steel reinforcement yields, followed by rupture of GFRP. This failure mode is seen in all RF group. This failure mode is where the composite action between concrete and FRP is maintained.

Figure 32 Flexural failures in group RF1, RF2 and RF3

5. Analytical Investigation

5.1 General

Although the technique of externally bonded reinforcement is quite new, there are already several codes and guidelines available for engineers for planning a retrofitting project.

The use of FRP as external reinforcement in strengthening RC structure requires the development of design procedures that ensure adequate safety. Several failure modes could develop in retrofitted beams using FRP i.e. compression failure before or after steel yielding, FRP rupture before or after steel yielding and loss of composite action. Most of the design guidelines overcome the above failure modes in the design in different ways. The flexural capacity in guidelines comes from three parts: concrete, steel and FRP.

Ultimate limit state (ULS) analysis of RC members strengthened with FRP is used in all guidelines and relies on the following fundamental assumptions:

- A section plane before bending remains plane after bending.
- No relative slip between external FRP and concrete.
- Take into consideration the effect of initial load prior to strengthening; this is by Considering the initial strain distribution in the calculations.

The position of the neutral axis is computed by means of a force equilibrium equation along the beam axis. The moment capacity M_d of the strengthened member can then be calculated using a moment equilibrium equation, see Figure 33.

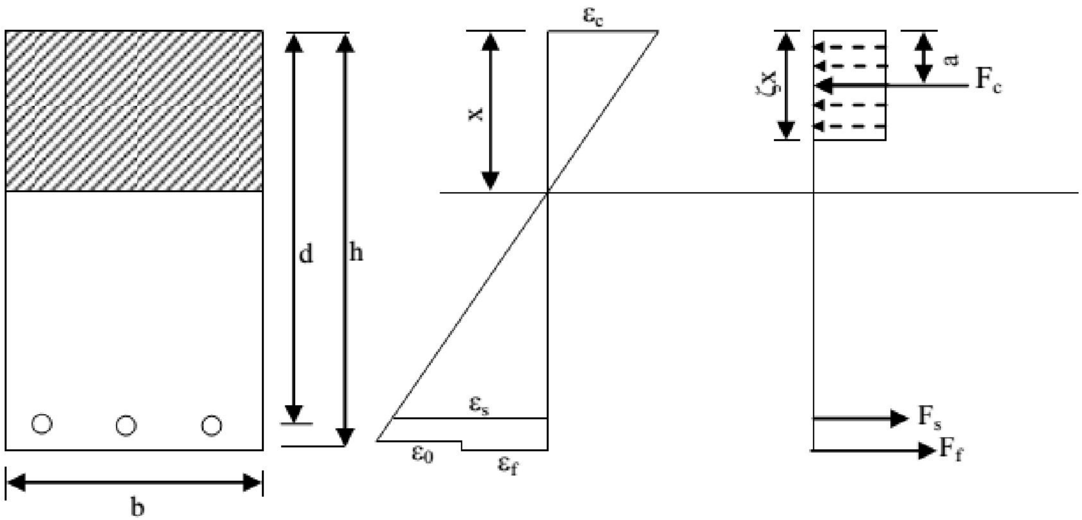


Figure 33 Stress and strain diagram for a cross-section of a rectangular beam

The nominal moment capacity corresponding to the two failure modes can be expressed by:

- Steel yielding and FRP rupture:

$$M_d = A_s f_y (d - a) + A_f E_f \varepsilon_{fu} (h - a) \quad (1)$$

- Steel yielding and concrete crushing:

$$M_d = A_s f_y (d - a) + \left(\left(\frac{h-x}{x} \right) \varepsilon_{cu} - \varepsilon_0 \right) A_f E_f (h - a) \quad (2)$$

Using hand calculation moment capacity has been done .As long as steel yielding and FRP rupture happening is concerned, by using equation one moment capacity and load at yield point can be computed.

Substituting on equation one moment capacity of the control beam is 4.28KNm and load capacity at yielding point is 21.38KN.

For retrofitted beam moment capacity is 8.15KNm, 12.01KNm and 15.87KNm for RF1, RF2 and RF3 respectively.

Calculating load capacity at yielding point is 40.38KN, 59.69KN and 79KN for RF1, RF2 and RF3 series respectively.

The finite element method is a very powerful for analyzing the behavior of concrete members without having to go to expensive and time consuming work in the laboratory. ANSYS 15.0 was employed to simulate the flexural and behaviour of the beam by finite element method.

5.2 Modeling of Materials

5.2.1 Concrete

The Solid65 element was considered to model the concrete. The solid element has eight nodes with three degrees of freedom at each node-translation in the nodal x, y and z directions.

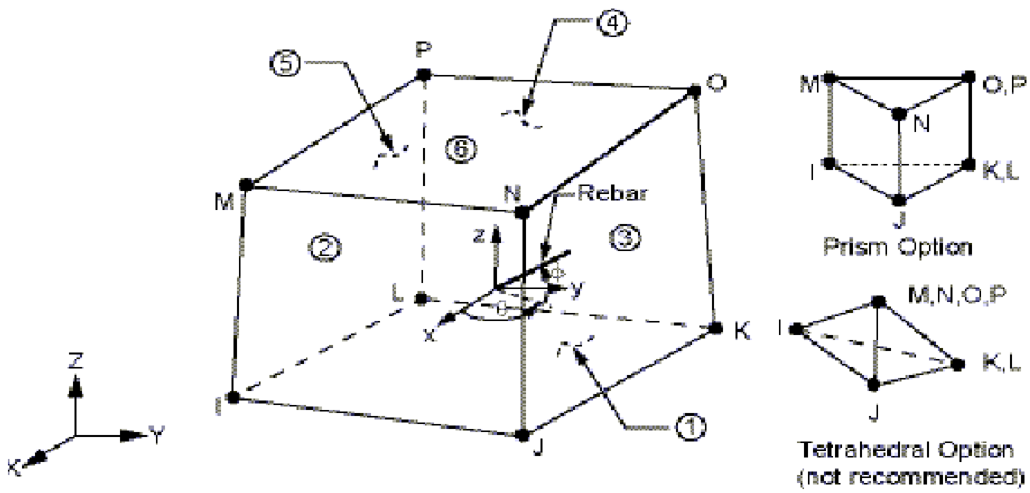


Figure 34 Solid65-3D reinforced concrete solid

For concrete, ANSYS requires an input data for material properties, which are Elastic modulus (Ec), ultimate uniaxial compressive strength (fc'), ultimate uniaxial tensile strength, Poisson's ratio (v).The modulus of elasticity and tensile strength of the concrete are calculated from equations 1 and 2 (CAN/CSA-A23.3-04, 2004). A Poisson's ratio of 0.18 is assigned to the concrete (Wight and MacGregor, 2009). To properly model the concrete, the considered model for concrete includes linear and multilinear material properties in addition to the concrete model defined in ANSYS (Kachlakev et al., 2001; Wolanski, 2004) as describes in the rest of this section.

The simplified compressive stress-strain curve for the concrete model was obtained by applying equations 4 to 8 to form the multi-linear stress-strain curve as plotted in Figure 35 (Wolanski, 2004; Wight and MacGregor, 2009).

$$E_c = 4500\sqrt{f'_c} \qquad (3)$$

$$f_r = 0.6\sqrt{f'_c} \qquad (4)$$

$$f_c = \varepsilon E_c \qquad \text{if} \qquad 0 \le \varepsilon \le \varepsilon_1 \qquad (5)$$

$$f_c = \frac{\varepsilon E_c}{1 + \left(\frac{\varepsilon}{\varepsilon_o}\right)^2} \qquad \text{if} \qquad \varepsilon_1 \le \varepsilon \le \varepsilon_o \qquad (6)$$

$$f_c = f'_c \qquad \text{if} \qquad \varepsilon_o \le \varepsilon \le \varepsilon_{cu} \qquad (7)$$

$$\varepsilon_0 = \frac{2f'_c}{E_c} \qquad (8)$$

where Ec, fr, f′c, ε_1, fc, ε , and ε_0 are the concrete modulus of elasticity (MPa), the concrete tensile strength (MPa), the concrete ultimate compressive strength (MPa), the strain at the end of the linear part up to fc= 0.3f′c (Wight and MacGregor, 2009), the concrete compressive stress at strain ε, the strain at stress fc, and the strain at maximum concrete strength, respectively.

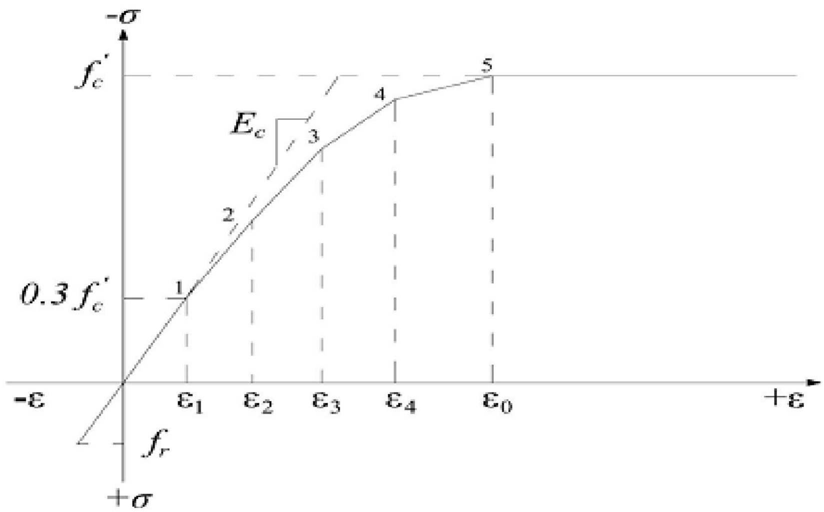


Figure 35 Simplified concrete compressive stress-strain curves.

5.3 Steel Reinforcements

The Link8 element was selected to model steel reinforcements. In the finite element models, steel bars were assumed to be made of an elastic-perfectly plastic material and the behaviour in tension and compression was identical. Poisson's ratio of 0.3 was used, and the elastic modulus, Es of 200,000 MPa. (Wight and MacGregor,2009).

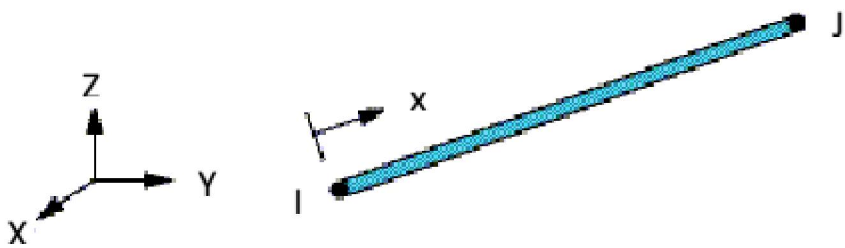


Figure 36 link8- 3D spar

5.4 FRP Strips

The Solid45 element was employed to model the FRP strips. A linear-elastic material behaviour up to failure was assigned to the FRP elements with a modulus of elasticity of 21GPa and a tensile strength of 772.95 MPa, an ultimate strain of 0.037. A Poisson's ratio of 0.28 was considered for this material

Table 9The materials used for finite element analysis and their properties

PROPERTIES	STRUCTURAL STEEL	CONCRETE	GFRP
Density (kg/m^3)	7850	2300	243.5
Youngs Modulus (Gpa)	200	43.4	21
Poissons ratio	0.3	0.18	0.28
Ultimate tensile strength (Mpa)	460	3.95	772.95

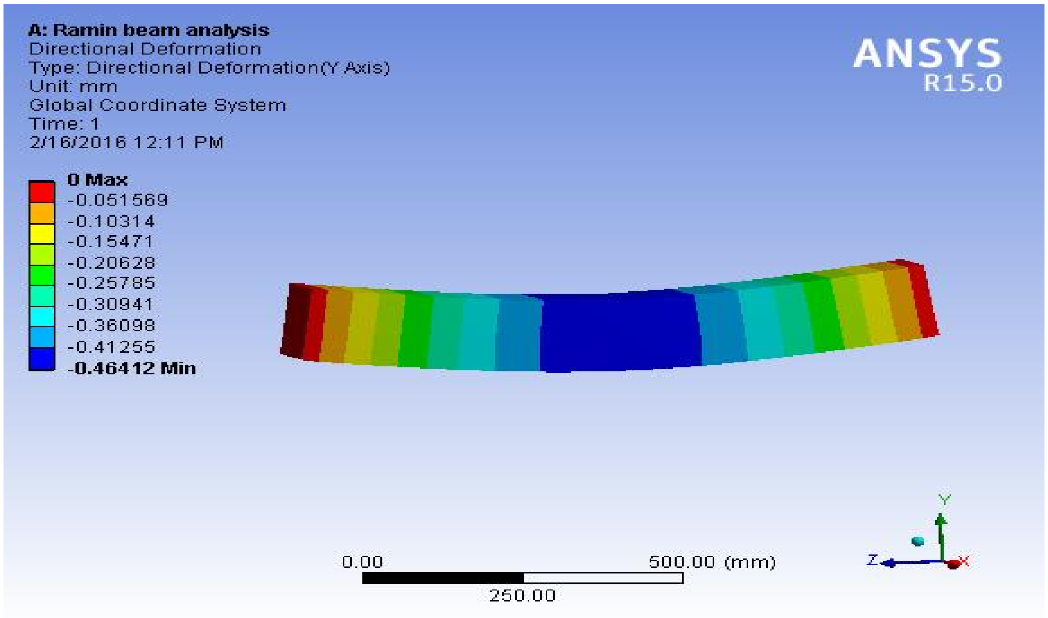


Figure 37 ANSYS result for deflection

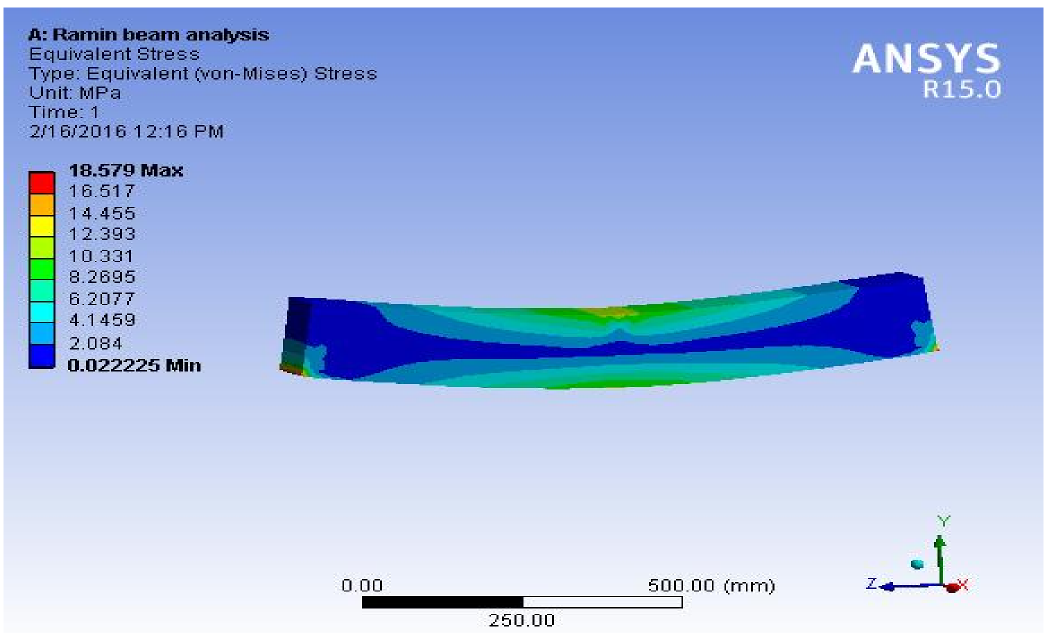


Figure 38 ANSYS result for equivalent stress

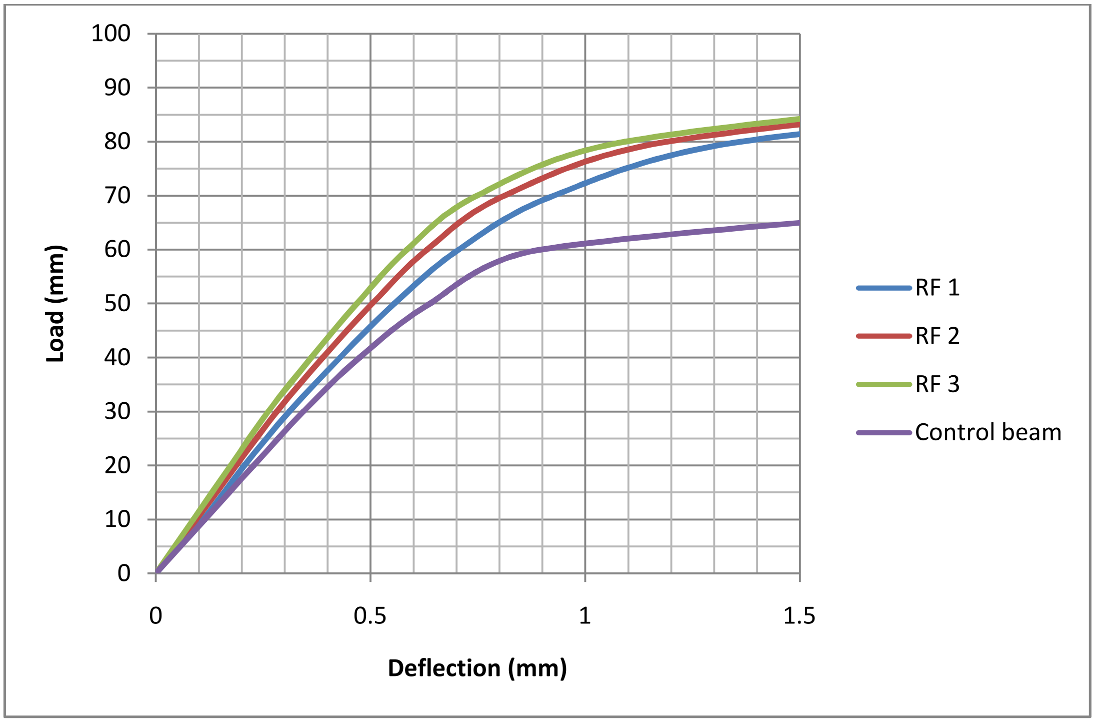


Figure 39 ANSYS result for Load Vs deflection of control beams in group RF

Comparison between the load-deflection results obtained from finite element analysis for control and retrofitted specimens shows that the yield load and the ultimate load has significantly increased for the retrofitted specimen. The yield load of GFRP specimens RF1, RF2 and RF3 are 10.5%, 16.67% and 23.07% more than the control specimen.

6. Comparison of Analytical and Experimental Results

In this section comparative study of numerical and experimental results will be discussed. Experiments and simulations showed that retrofitting can increase load capacity and stiffness for flexural capacity of beams.

The flexural capacities of flexure strengthened beams were improved by more than 50% by applying the GFRP as externally reinforcement.

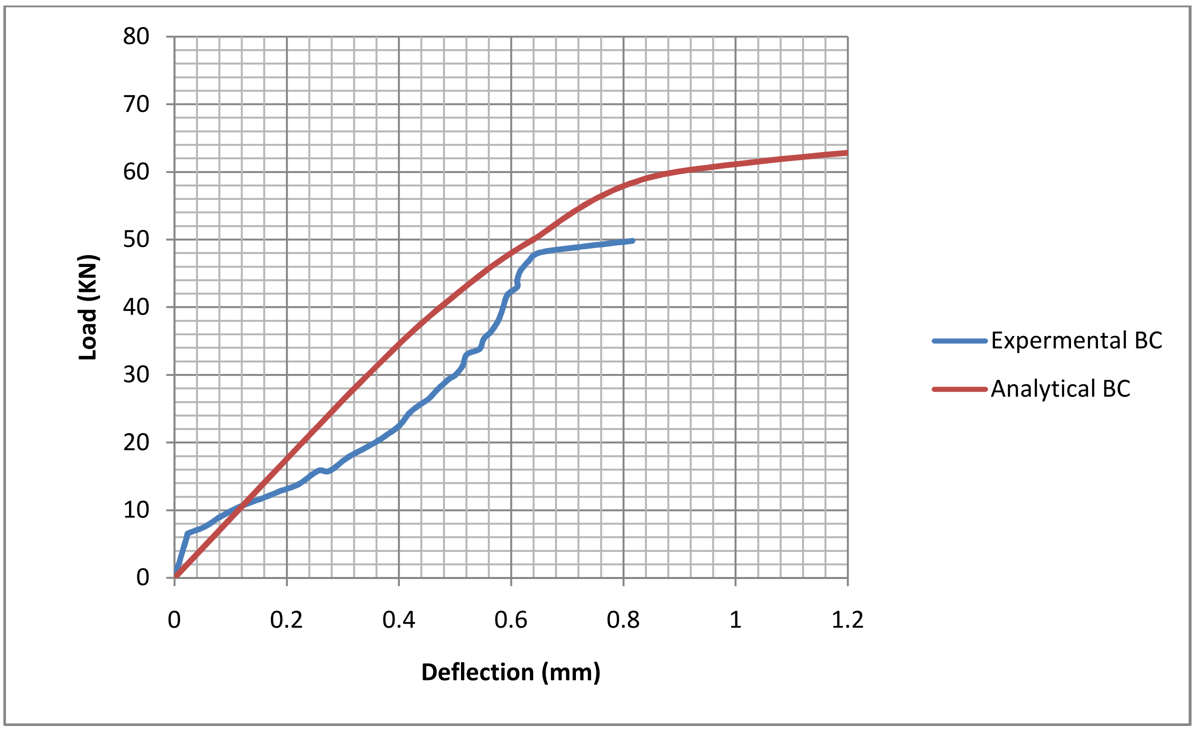


Figure 40 Comparison between load-deflection curves for control beam of experimental and analytical results

Comparison between the load deflection results of control beam obtained from finite element analysis and that from the experimental shows that the finite element analysis results are more than the experimental results. The yield load and ultimate load are 5% and 20.5% more than the experimental results respectively.

For the retrofitted beams there is a significant change in yield load and ultimate load capacity of beams as shown in figures below.

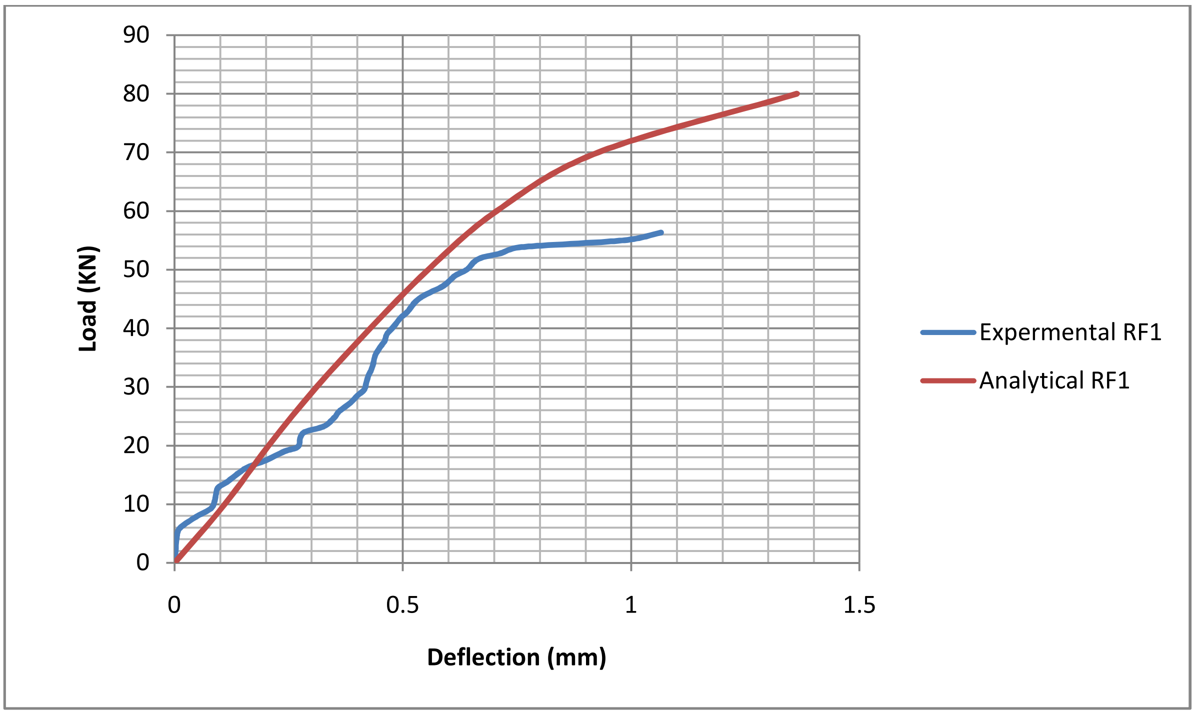


Figure 41 Comparison between load-deflection curves for RF1 beam of experimental and analytical results

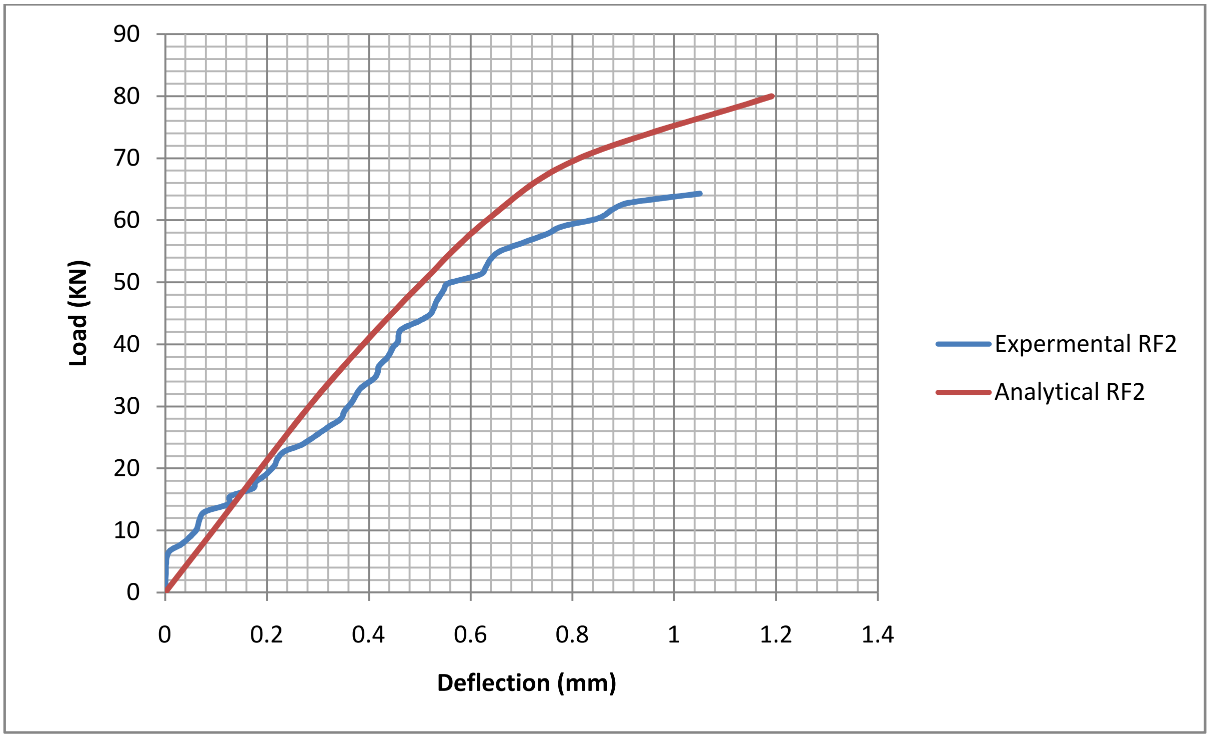


Figure 42 Comparison between load-deflection curves for RF2 beam of experimental and analytical results

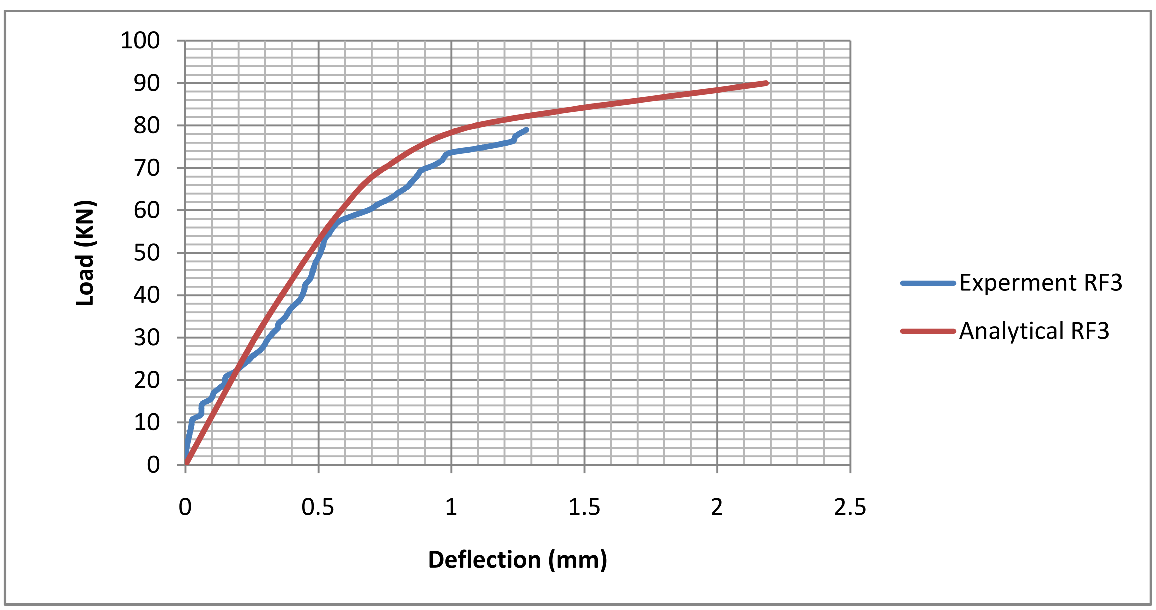


Figure 43 Comparison between load-deflection curves for RF3 beam of experimental and analytical results

There are several possible causes for the differences between the experimental data and the finite element analysis. One is, as for the control beam, the assumed perfect bond between concrete and steel reinforcement. Another reason is due to the estimation of the behaviour of the interface between GFRP and concrete. This may lead to the overestimation of the stiffness and capacity of the reinforced concrete structural element.

Hand calculation result of the determined value of load capacity at yielding point is closer to the finite element analysis but it far to the experimental results this may raise due to some unaccountable factors within the moment capacity determination., like interface consideration between GFRP and concrete.

7. Conclusions and Recommendations

7.1 Conclusions

In this research experimental study of flexural behaviour of RC beams strengthened with externally by FRP sheets were studied and numerical study is carried out using the finite elements software ANSYS.

The finite element method is a useful for the understanding of the behaviour of reinforced concrete beams retrofitted with FRP. Experimental tests are needed to provide input data to the model and for the purpose of verification of simulation results. When the model has been validated it can be used for parametric studies to investigate the influence of various parameters. The numerical results show good agreement with the experimental values in general.

From the conducted experimental and analytical study on RC beams strengthened in flexure, with externally bonded GFRP reinforcement using epoxy adhesive, the following can be concluded:

- The FRP material can be used for structural use, rather to its fabricated purpose of plaster reinforcement. The material is capable of increasing flexural capacity.
- Experiments and simulations showed that retrofitting can increase load capacity and stiffness for flexural capacity of beams.
- The flexural capacities of the flexure strengthened beams were improved by more than 50% by applying the GFRP as externally reinforcement.
- The layers of the fibres were found to have an important effect, especially where 3 layers were applied. There was a greater strengthening effect and better control of the flexural crack propagations.
- This method is relatively easy for construction and handling, can be used effectively for strengthening RC beams that require increased in flexural capacity.

- Comparison between the load deflection results obtained from the experimental and finite element analysis shows that the finite element analysis results are more than the experimental results. The yield load and ultimate load are 5% and 20.5% more than the experimental results respectively.
- The failure modes observed in the retrofitted beams were as the steel reinforcement yields, it was followed by rupture of GFRP.
- Comparison between the load-deflection results obtained from finite element analysis for control and retrofitted specimens shows that the yield load and the ultimate load has significantly increased for the retrofitted specimen. The yield load of GFRP specimens RF1, RF2 and RF3 are 10%, 16.67% and 23.07% more than the control specimen.

7.2 Recommendation

The experimental program has shown that the FRP plate retrofitting system enhances the capacity of deficient concrete beams. There are, however, many environmental factors involved during the life span of a retrofitted structure that need more attention. They include seasonal temperature variation, degradation of material properties, creep and so on. The durability of FRP reinforced beams under these conditions should be investigated.

The material is capable of increasing flexural capacity. However it should be noted that the compression side of beam needs to be both retrofitted for a required result by attaining the ductility at interest.

Further refinement of the numerical model could be of interest. Especially concerning the modeling of the FRP-concrete interface, there is still a need for further development.

There are many design guidelines available for retrofitted structures. These guidelines, however, have different criteria for predicting debonding. This indicates a lack of fundamental understanding of the phenomenon, and further research is needed.

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APPENDICES

Appendix 1

VITRULAN

Plaster Reinforcing Fabric

SDA.4412

Premium Quality

Application
- To reinforce all types of plaster, applied externally or internally
- For load-bearing construction and general renovation work
- Preventing cracks in plastering
- As reinforcing base for gypsum or wooden surface

Advantages
- Excellent adhesion with all types of plaster
- Non-corrosive
- High alkali resistant
- High force absorbtion at low elongation
- High tensile strength in both directions (warp and weft)
- Easy to handle and to install
- No need for nails or hammers

Installation
- Wet the surface properly, apply a thin base coat of plaster and then press the fabric gently into the wet plaster using a trowel. Then apply the final coat whilst the initial is still wet.

Material
- Mesh fabric is made of pure textile fiberglass (E-glass)
- Highly resistant to alkali according to the ETAG 004 approval guideline drawn up by EOTA
- Environmental performance of the product which is certified by the manufacturer's EPD – Environmental Product Declaration.

THIRD-PARTY VERIFIED EPD Institut Bauen und Umwelt e.V.

Technical information

Weight*	168 g/m²	
Mesh Size	6.0 x 6.0 mm	
Thickness	0.69 mm	
Tensile Strength**		
Warp	2000 N per 5 cm	
Weft	2200 N per 5 cm	
Elongation**		
Warp	3.7 %	
Weft	3.7 %	
Alkali Resistance*** Tensile Strength after 28 days conditioning in 3 Ion solution		
Warp	≥ 50% and	≥ 1000 N per 5 cm
Weft	≥ 50% and	≥ 1000 N per 5 cm

* Test Method according to DIN 53 854
** Test Method according to DIN EN ISO 13934-1
*** Test Method according to ETAG 004 guide line

High resistance to alkali - ETAG 004 approved according to EOTA (www.eota.eu)

ETAG 004 = European Technical Approval Guide line drawn up by the EOTA = EUROPEAN ORGANISATION FOR TECHNICAL APPROVALS

VITRULAN is conform with ASTM D-76, ASTM D-579, ASTM D-5035, ASTM E-2098

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Appendix 2

Construction

Product Data Sheet
Edition 01.2012/v1
CSC Master Format™ 03 05 00
Sikadur® 32 Hi-Mod

Sikadur® 32 Hi-Mod

High-Modulus, High-Strength, Epoxy-Based Protective Coating and Bonding Adhesive

Description

Sikadur® 32 Hi-Mod, is a multi-purpose, two-component, solvent-free, moisture-insensitive, structural epoxy adhesive and protective coating.

Where to Use

- Protective coating for reinforcing steel.
- Bond fresh, plastic concrete to hardened concrete and steel.
- Grout bolts, dowels, and pins etc.
- Grout horizontal cracks in structural concrete and wood by gravity feed.
- Structural adhesive for concrete, masonry, metal, wood, etc.

Advantages

- High-build, chemically resistant and protective coating.
- Super-strength bonding/grouting adhesive.
- Insensitive to moisture before, during and after cure.
- Excellent adhesion to most structural materials.
- Easy to mix: 1:1 ratio.
- Easy to use for bonding/grouting applications.
- Free of service-inhibiting polysulfides.
- Fast initial set; rapid gain to ultimate strengths.
- USDA-approved for use in food plants.
- Meets ASTM C881, Type I, II and V, Grade 2, Class B and C, epoxy resin adhesive.
- Ministère des Transports du Québec acceptance.

Technical Data

Packaging	10 L (2.6 US gal.) unit		
Colour	Concrete Grey		
Yield	1 L = approx. 2 m² (1 US gal. = approx. 80 ft²)		
Shelf Life	2 years in original, unopened packaging. Store dry at 5 to 32°C (41 to 89°F). Condition product at 18 to 30°C (65 to 86°F) before using.		
Mix Ratio	A:B = 1:1 by volume		
Contact Time	**4°C (39°F)***	**23°C (73°F)***	**32°C (89°F)***
	14 to 16 hrs	3 hrs 30 min to 4 hrs	1 hr 30 min to 2 hrs

Properties at 23°C (73°F) and 50% R.H.

Viscosity	2800 cps		
Pot Life, 318 g (11.2 oz)	30 - 38 min		
Compressive Strength ASTM D695, MPa (psi)			
	4°C (39°F)*	**23°C (73°F)***	**32°C (89°F)***
8 hrs	-	-	7 (1015)
16 hrs	-	17 (2466)	31 (4498)
1 day	-	32 (4643)	44 (6384)
3 days	5 (725)	56 (8125)	57 (8270)
7 days	50 (7255)	66 (9576)	57 (8270)
14 days	56 (8125)	66 (9576)	57 (8270)
28 days	60 (8706)	66 (9576)	57 (8270)

* Product cured and tested at the temperatures indicated.

Modulus of Elasticity ASTM D695		
28 days	3.03 GPa (4.4 x 10⁵ psi)	
Tensile Properties ASTM D638		
14 days	Tensile strength	33 MPa (4788 psi)
	Elongation at break	1.9%
	Modulus of elasticity	2.2 GPa (3.2 x 10⁵ psi)
Flexural Properties ASTM D790		
14 days	Modulus of rupture	51 MPa (7400 psi)
	Tangent modulus of elasticity in bending	3.24 GPa (4.7 x 10⁵ psi)
Shear Strength ASTM D732		
14 days	41 MPa (5949 psi)	
Water Absorption ASTM D570		
7 days	2 hrs boil	0.7%

Construction

Deflection Temperature ASTM D648		
14 days	Fiber stress loading = 1.8 MPa (261 psi)	49°C (120°F)
Bond Strength ASTM C882		
14 days	Plastic concrete to hardened concrete	13 MPa (1886 psi)
	Plastic concrete to steel	13 MPa (1886 psi)

Product properties are typically averages, obtained under laboratory conditions. Reasonable variations can be expected on-site due to local factors, including environment, preparation, application, curing and test methods

How to Use

Surface Preparation

Substrate must be clean and sound. It may be dry or damp, but free of standing water. Remove dust, laitance, grease, curing compounds, impregnations, waxes, foreign particles and disintegrated materials.
Concrete: Sandblast or use other approved mechanical methods.
Steel: Sandblast to white-metal finish (SP-10).

Mixing

Pre-stir each component then proportion equal parts by volume of component A and component B into a clean pail. Mix thoroughly for 3 minutes with paddle on low-speed drill (300-450 rpm) until blend is a uniform colour. Mix only that quantity that can be applied within its pot life.

Application

To protect steel reinforcing: Apply two coats of Sikadur® 32 Hi-Mod by brush or spray. Allow first coat to become tack-free. Apply second coat prior to application of repair mortar/concrete.
To bond fresh concrete to hardened concrete: Apply by brush, roller, broom or spray. Place fresh concrete while Sikadur® 32 Hi-Mod is still tacky. If coating becomes glossy and loses tackiness, remove any surface contaminates then recoat with additional Sikadur® 32 Hi-Mod and proceed.
To anchor bolts, dowels and pins: Use neat. For efficient transfer of stress, the holes should be not greater than 6 mm (1/4 in) in diameter than the bar, pin or rod to be embedded. Depth of embedment is typically 10 to 15 times the bar diameter.
To gravity feed cracks: Pour neat material into "V"-notched crack. Continue placement until completely filled. Seal underside of slab prior to filling if cracks reflect through.

Clean Up

Collect with absorbent material. Dispose of in accordance with local disposal regulations. Uncured material can be removed with Sika® Equipment Cleaner. Cured product can only be removed mechanically.

Limitations

- Do not use as a bonding agent with set accelerated mortars, e.g. SikaQuick® 1000, SikaQuick® 2500, SikaTop® 122 PLUS Winter Grade and SikaTop® 123 PLUS Winter Grade. Consult Sika Canada Technical Services.
- Minimum application temperature: 4°C (39°F).
- Product is a vapour barrier after cure.
- Do not thin with solvents.

Health and Safety Information

For information and advice on the safe handling, storage and disposal of chemical products, users should refer to the **most recent Material Safety Data Sheet** containing physical, ecological, toxicological and other safety-related data.

KEEP OUT OF REACH OF CHILDREN
FOR INDUSTRIAL USE ONLY

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